

Geotechnical Engineering of Dams

2nd edition

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A BALKEMA BOOK

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Cover illustration: Hinze Dam at completion of Stage 3 construction. Courtesy of The Hinze Dam Alliance (comprising Seqwater, SKM, Thiess and URS).

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Author Biographies



Robin Fell is Emeritus Professor of Civil Engineering at the University of New South Wales and also works as a peer review consultant. He has 48 years of experience in geotechnical engineering of dams, landslides and civil and mining projects in Australia, New Zealand, the Pacific, North and Central America, and Asia. He has worked on over 100 dams worldwide and has been involved in all aspects of planning, site investigation, design and construction of embankment and concrete gravity dams.



Patrick MacGregor is a Consulting Engineering Geologist with more than 50 years experience in the assessment of geological constraints for major civil engineering projects in a number of countries. He has been involved in dam investigation, design and construction and particularly worked on hydroelectric developments at all stages from inception to operation.



David Stapledon spent many years on the investigation of large dam construction sites in various countries. He was a Professor of Engineering Geology at the University of South Australia (1964–1993) and worked as a Consultant in Engineering Geology, contributing to major dam projects in Australia, New Zealand and South East Asia. He has more than 50 years of experience and was awarded the John Jaeger Memorial Medal for Contributions to Geomechanics in 1995.



Graeme Bell has worked in dam engineering since 1962. His role has varied from providing the full technical input, design management and construction advice for new dams to the preparation of complex structural analyses of existing dams. From 1979, he has acted as an independent reviewer on many dam projects, mainly in Australia, but also in several overseas locations.



Mark Foster has 20 years of experience in dam engineering and geotechnical engineering. This has involved a wide variety of projects including dam safety reviews, design of dam upgrade projects and dam safety risk assessments for embankment and concrete dams. He has a particular interest in the assessment of piping and internal erosion of embankment dams which was the topic of his doctoral research studies at the University of New South Wales.

Chapter 1

Introduction

1.1 OUTLINE OF THE BOOK

The book sets out to present a state of practice of the geotechnical engineering of embankment dams and their foundations and the geotechnical engineering of the foundations of concrete dams.

It assumes that the reader is trained in Civil Engineering or Engineering Geology, with knowledge of soil and rock mechanics.

There is an emphasis on the assessment of existing dams, as well as investigation, design and construction of new dams. We have set out to give the background to design methods, as well as the methods themselves, so the reader can develop a proper understanding of them. The book is largely written about large dams, which ICOLD (1974) define as:

Large Dam: A dam which is more than 15 metres in height (measured from the lowest point in the general foundations to the crest of the dam), or any dam between 10 metres and 15 metres in height which meets one of the following conditions:

- the crest length is not less than 500 metres;
- the capacity of the reservoir formed by the dam is not less than one million cubic metres;
- the maximum flood discharge dealt with by the dam is not less than 2000 cubic metres per second;
- the dam is of unusual design.

This is the second edition of Fell, MacGregor, Stapledon and Bell, Geotechnical Engineering of Dams (2005). Almost all Chapters in that book have been extensively revised to reflect the current state of practice and the knowledge the authors have gained since that book was written.

The most significant changes are in regards to:

- Chapters 8 and 9 on internal erosion and piping of dams and their foundations. There has been extensive research carried out in Australia, Europe and North America leading to a much better understanding of the mechanics of the processes which are now able to be applied in practice.
- Appendix A which along with Chapter 9 gives a detailed description of methods for assessing the likelihood of failure of embankment dams by internal erosion

Table 1.1 Embankment dam zones description and function.

Zone	*Description*	*Function*
1	Earthfill ("core")	Controls seepage through the dam
2A	Fine filter (or filter drain)	(a) Controls erosion of Zone 1 by seepage water, (b) Controls erosion of the dam foundation (where used as horizontal drain), (c) Controls buildup of pore pressure in downstream face when used as vertical drain
2B	Coarse filter (or filter drain)	(a) Controls erosion of Zone 2A into rockfill, (b) Discharge seepage water collected in vertical or horizontal drain
2C	(i) Upstream filter (ii) Filter under rip rap	Controls erosion of Zone 1 into rockfill upstream of dam core Controls erosion of Zone 1 through rip rap
2D	Fine cushion layer	Provides uniform support for concrete face; limit leakage in the event of the concrete face cracking or joints opening
2E	Coarse cushion layer	Provides uniform layer support for concrete face. Prevents erosion of Zone 2D into rockfill in the event of leakage in the face
1–3	Earth-rockfill	Provides stability and has some ability to control erosion
3A	Rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam. Prevents erosion of Zone 2B into coarse rockfill
3B	Coarse rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam
4	Rip rap	Controls erosion of the upstream face by wave action, and may be used to control erosion of the downstream toe from backwater flows from spillways

and piping. The authors have been at the forefront of developing and application of these methods over a period of 15 years.

- Chapter 10 on embankment dams, their zoning and design for control of seepage and internal erosion and piping. This has been modified to better separate the requirement of zoning to control seepage and pore pressures for stability, and to control internal erosion and piping.
- Chapter 12 on design of embankment dams for earthquake where there have been significant advances in the state of the art in the last ten years.

1.2 TYPES OF EMBANKMENT DAMS AND THEIR MAIN FEATURES

There are several types of embankment dam. The designs have varying degrees of in-built conservatism, usually relating to the degree to which seepage within the dam is controlled by provision of filters and drains, the use of free draining rockfill in the embankment and the control of foundation seepage by grouting, drainage and cutoff construction.

Table 1.1 lists the common embankment dam zones and their functions. The zone numbering system shown is used throughout this book. There is no universally adopted

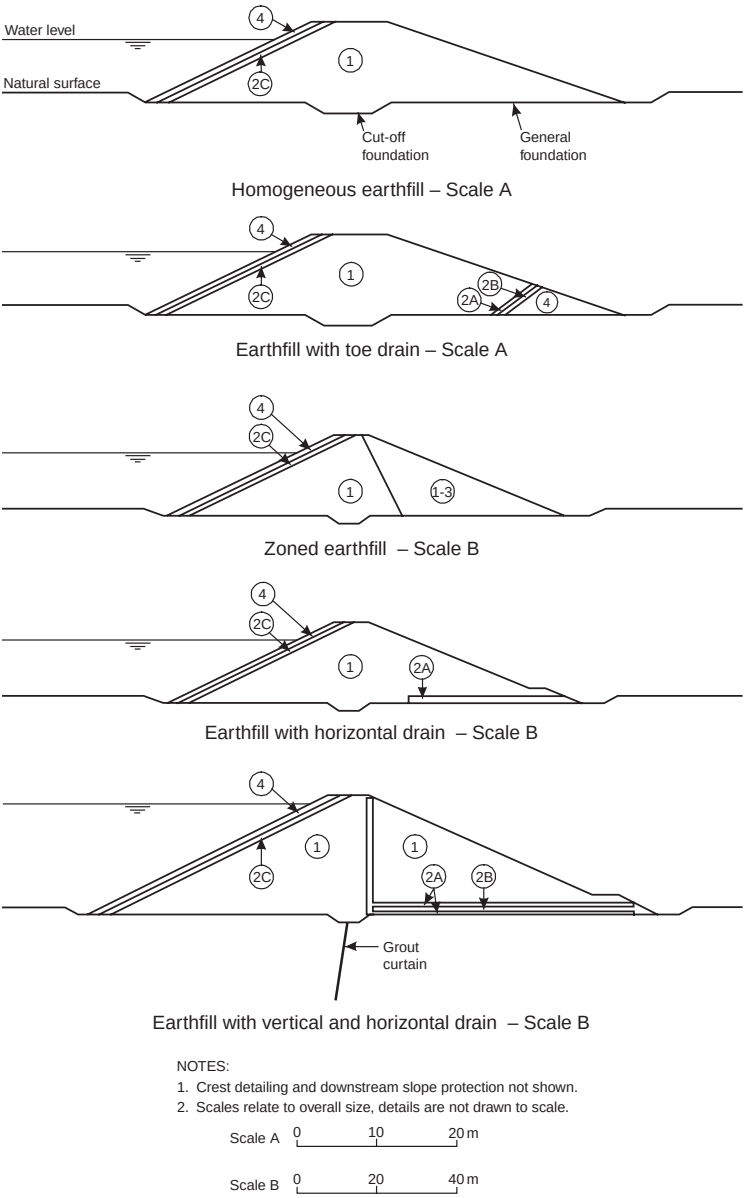

Water level
Natural surface
4
1
2C
Cut-off foundation
General foundation
Homogeneous earthfill – Scale A
2B
2A
Earthfill with toe drain – Scale A
1-3
Zoned earthfill – Scale B
Earthfill with horizontal drain – Scale B
Grout curtain
Earthfill with vertical and horizontal drain – Scale B
NOTES:
1. Crest detailing and downstream slope protection not shown.
2. Scales relate to overall size, details are not drawn to scale.
Scale A 0 10 20 m
Scale B 0 20 40 m

Figure 1.1 Schematic cross sections of typical earthfill dams.

numbering system for zones in embankment dams. Some dams will have several rockfill zones to accommodate the materials available from the quarry and excavations required for the spillway.

Figures 1.1 and 1.2 show schematic cross sections of the most common types of embankment dams now being constructed. Figure 1.3 shows some earlier types of

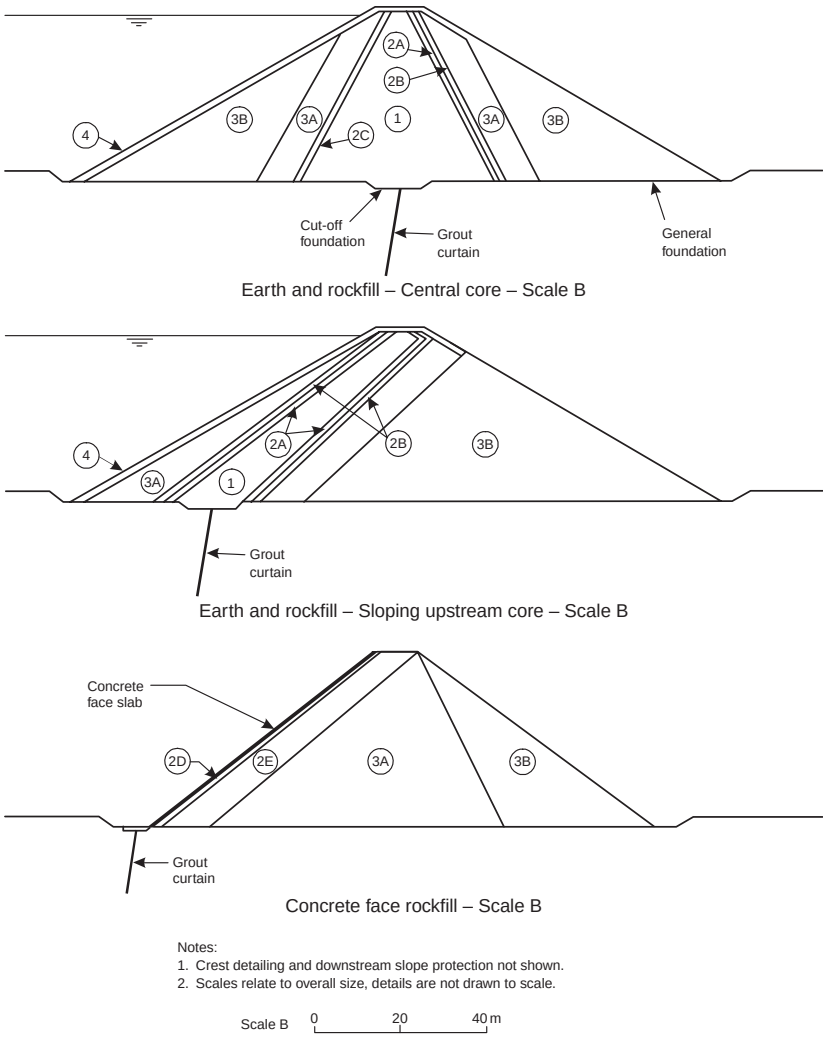


Figure 1.2 Schematic cross sections of typical earth and rockfill and concrete face rockfill dams.

dams which are no longer being built. There are other types of embankment dams, but these are not specifically covered in this book including:

- Concrete face earthfill;
- Asphaltic core rockfill;
- Bituminous concrete face earth and rockfill;
- Steel face rockfill;
- Geomembrane face earth and rockfill.

Table 1.2 describes the foundation treatment commonly used in embankment dams.

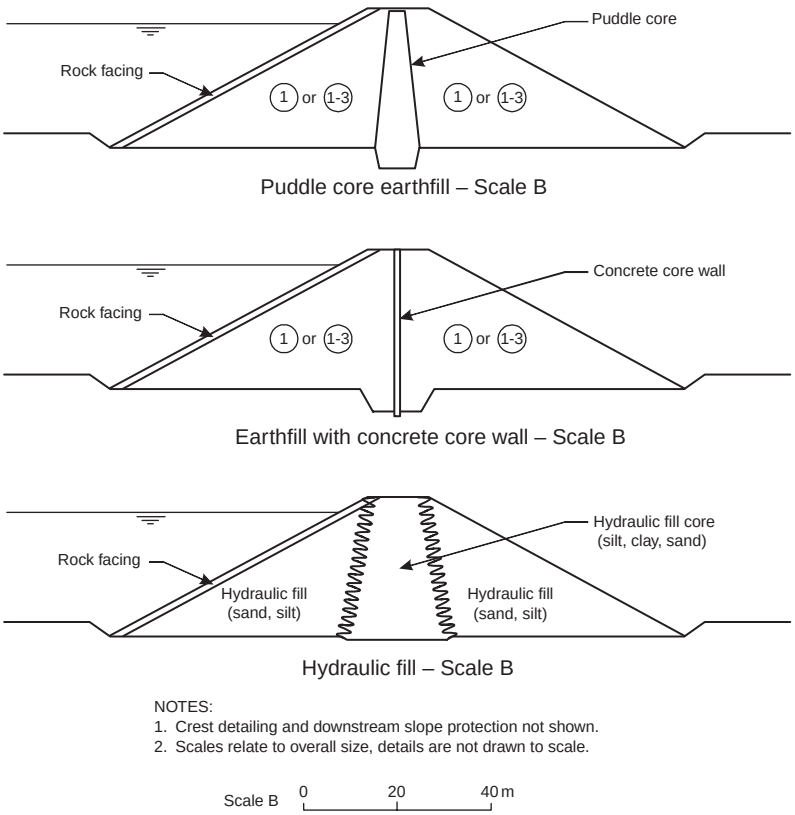


Figure 1.3 Schematic cross sections of some earlier dam types.

Table 1.2 Embankment dam foundation treatment.

Item	*Description*
General foundation excavation	Excavation of compressible and low strength soil and weathered rock as is necessary to form a surface sufficiently strong to support the dam and to limit settlement to acceptable values
Cutoff foundation excavation	Excavation below general foundation level to remove highly permeable and/or erodible soil and rock.
Curtain grouting	Drilling of holes into the foundation and injecting grout (usually cement slurry) under pressure to reduce the permeability of the rock
Consolidation grouting (also called "blanket grouting")	Grouting carried out in the upper part of the cut-off foundation to reduce permeability (of the rock)

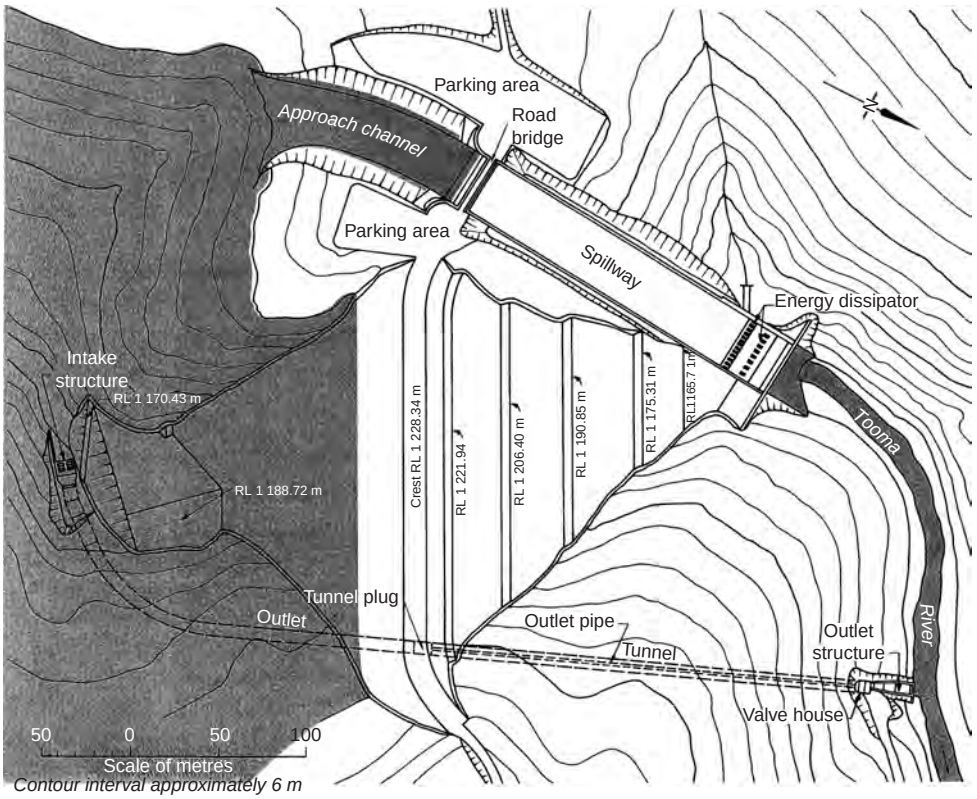


Figure 1.4 Plan of typical embankment dam and associated structures.

Figure 1.4 shows a fairly typical plan of an embankment dam showing the spillway, outlet tunnel (which is commonly used for river diversion during construction of the dam) and outlet structures.

1.3 TYPES OF CONCRETE DAMS AND THEIR MAIN FEATURES

Figures 1.5 and 1.6 show examples of concrete gravity and arch dams. Concrete gravity dams rely on the weight of the concrete to withstand the forces imposed on the dam. Concrete arch dams transpose these forces into the abutment foundation by the arching action and generally impose higher loads on the foundations.

Modern concrete gravity and arch dams have a gallery in the dam from which grout holes are drilled to reduce the rock mass permeability of the foundation. Grouting,

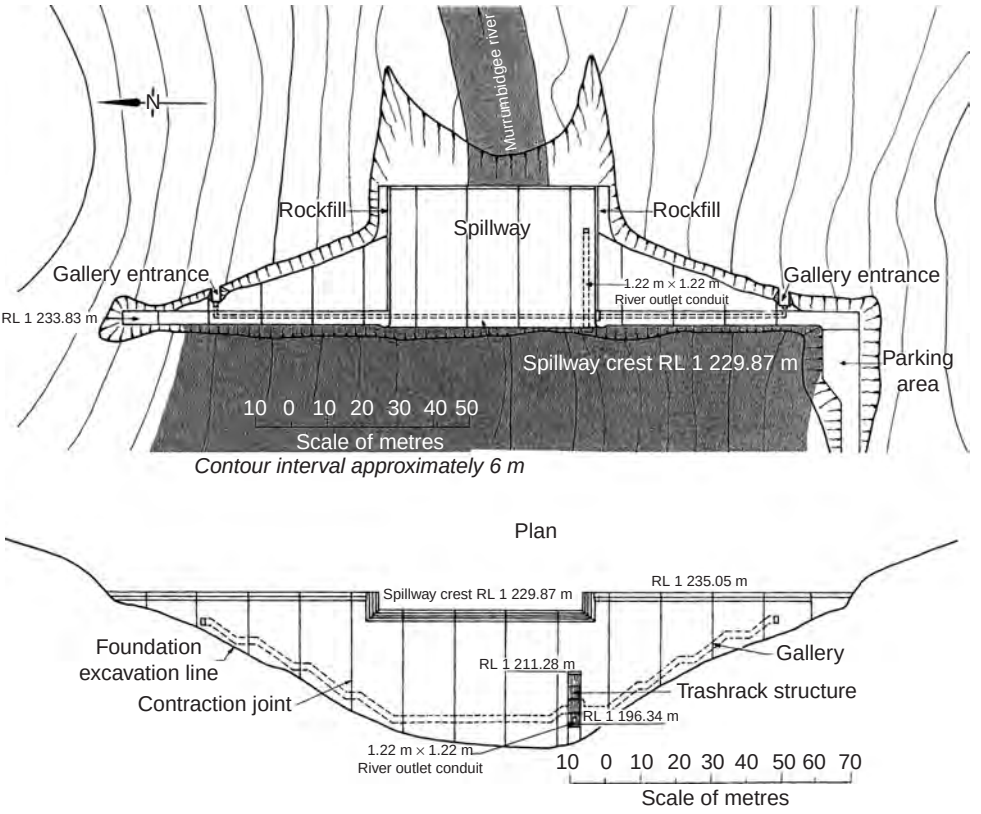


Figure 1.5a Typical concrete gravity dam plan and elevation.

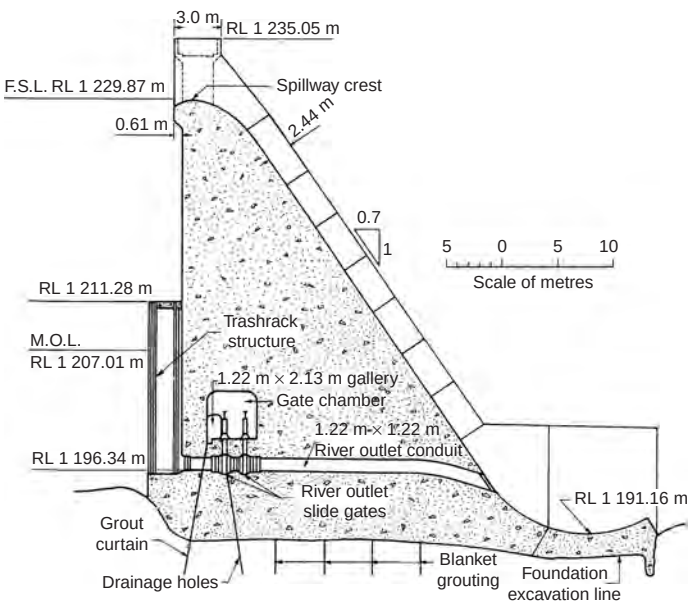


Figure 1.5b Typical concrete gravity dam cross section.

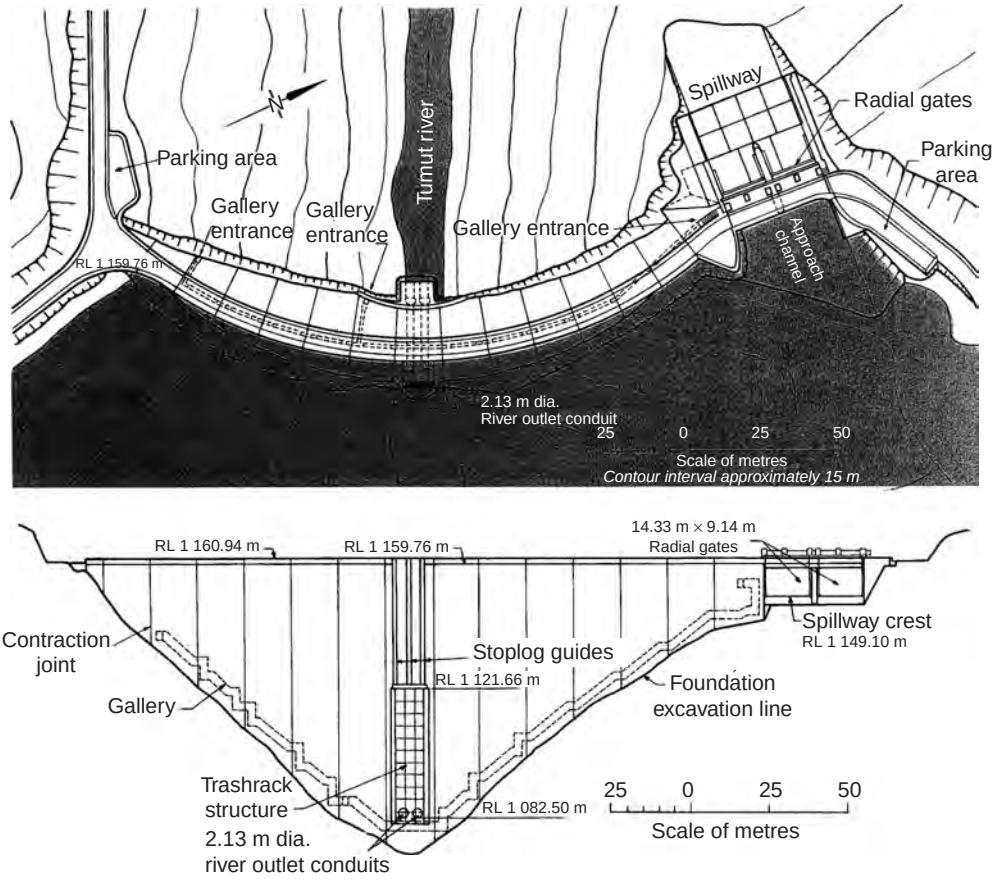


Figure 1.6a Typical concrete arch dam plan and elevation.

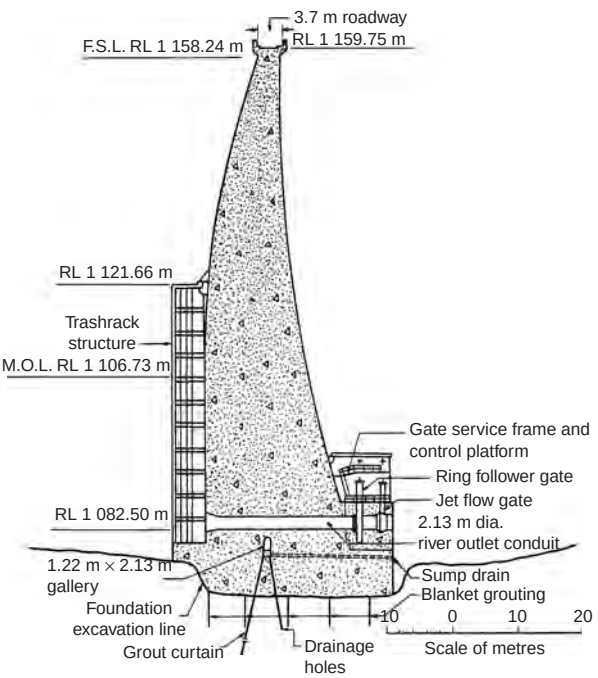


Figure 1.6b Typical concrete arch dam cross section.

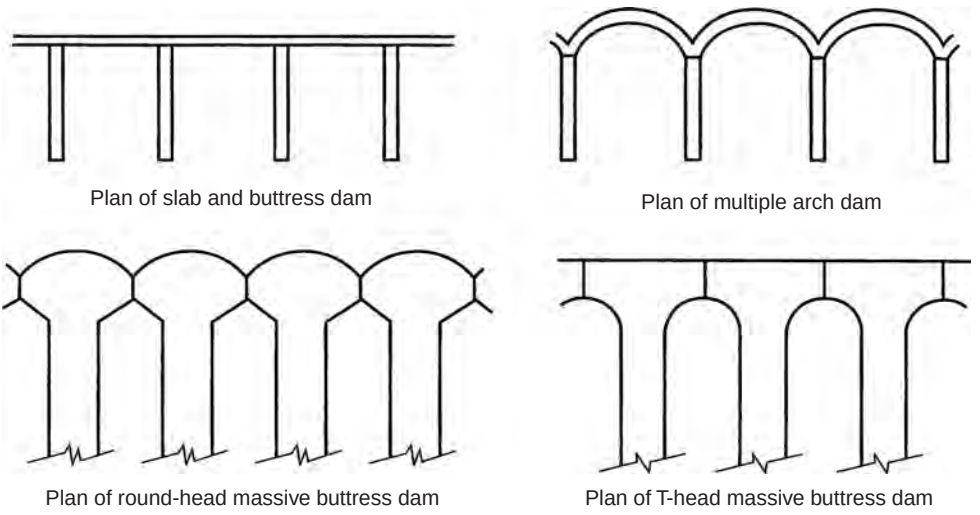

Plan of slab and buttress dam
Plan of multiple arch dam
Plan of round-head massive buttress dam
Plan of T-head massive buttress dam

Figure 1.7 Different types of buttress dam (ANCOLD 2000).

combined with drainage holes reduces the uplift pressures within the dam foundation and at the dam foundation contact.

Figure 1.7 shows different types of concrete buttress dams. Dependent on the configuration, these may impose large stresses on the dam foundation.

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Chapter 2

Key geological issues

Most dams and reservoirs are located at or near the bases of river valleys. The topography and geological situation at every site will have developed by the interaction of many geological and related processes during vast periods of time. The resulting geological structure at dam and reservoir sites can be complex and no two sites will be the same. Some of the processes which formed the site may still be active or may be reactivated by the project and capable of influencing the feasibility of its construction or operation. It is vital therefore, that sites are investigated using all appropriate knowledge and methods of classical geology. These are essential for good engineering geology (Baynes, 1999; Fookes, 1997; Fookes *et al.*, 2000; Hutchinson, 2001).

The engineering team responsible for construction of a dam at each site must be able to make reliable predictions about suitable construction methods and how the dam and its foundation will interact and perform, under every envisaged operating condition. The predictions will usually involve both judgments and quantitative analyses, based on data provided by the site investigation team. This data is provided as a geotechnical model, which consists of the following:

- a sufficiently detailed three-dimensional picture or model of the site geological situation, using geotechnical descriptive terms, and an assessment of the history of the site and the effects of any processes which are still active and
- adopted values of parameters for critical parts of the model, as required for the analyses.

This chapter introduces geotechnical descriptive terms and uses them while discussing processes found to be important in dam engineering. Defects or discontinuities and their modes of formation are defined and discussed first, because understanding of them is needed throughout the book. Some aspects of the valley-forming processes of weathering (breakdown) and erosion (removal) of rocks and soils are then covered in detail because they are the youngest and most active processes at most sites. For further study of these processes the reader is referred to Selby (1982), Bell (1983a), Hunt (1984), Fookes and Vaughan (1986) and Fookes (1997).

2.1 BASIC DEFINITIONS

The following definitions and descriptive terms are recommended, and are used throughout this book.

Material – a non-specific general term to describe any sample or body of soil or rock.

Substance – material which is effectively homogeneous in its engineering properties, but may be isotropic or anisotropic. The term "effectively" here means within the tolerances applicable to the project in hand.

Rock substance – a homogeneous cohesive aggregate of mineral grains which has such strength that it cannot be broken up or remoulded by hand when dry or when immersed in water.

Soil substance – a homogeneous aggregate of mineral grains, either non-cohesive or cohesive but which disintegrates or can be remoulded by hand, when immersed in water.

Defect – a discontinuity or break in continuity of a body of rock or soil substance.

Mass – any body or rock or soil which is not effectively homogeneous; it can comprise two or more substances without defects, or one or more substances with one or more defects.

In most situations the engineering behaviour of rock masses is dominated by the number, type and configuration of defects within them. The behaviour of soil masses is usually influenced, and sometimes dominated, by defects present in them.

2.2 TYPES OF ANISOTROPIC FABRICS

Any of the following types of fabric, developed uniformly within a body of rock or soil substance, will cause that substance to be anisotropic with respect to its engineering properties – strength, stiffness and permeability.

Bedding – layered or parallel arrangement of grains, developed during deposition as sediment

Foliation – layered or parallel arrangement of grains (often tabular or flakey in shape) developed either by viscous flow (in igneous rocks) or by pressure and heat (in metamorphic rocks)

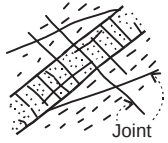
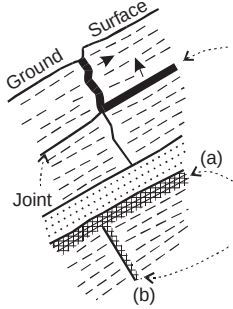
Cleavage – foliation in which many surfaces have developed along which the substance splits readily

Lineation – linear arrangement of (often elongated) grains, developed by viscous flow (in igneous rocks) or pressure with or without heat (in metamorphic rocks); the lines of grains may or may not lie within surfaces or layers of foliation.

If a body has any of these fabrics developed in a non-uniform way, that body is mass, by definition.

2.3 DEFECTS IN ROCK MASSES

Figure 2.1 illustrates and defines the most common and important types of defect met in rock foundations for dams. It also relates characteristics of the defect types to ways in which they were formed. The joints and two types of faults have developed directly from the stress histories of the rocks. Soil infill and extremely weathered seams usually occur along or within the above defect types, and hence most of them also result

	DEFECT NAME	DESCRIPTION	TYPICAL EXTENT/OCCURRENCE	FORMED BY
	Joint	Almost planar surface or crack, across which the rock usually has little tensile strength. May be open, air or water-filled, or filled by soil substance, or rock substance which acts as cement. Joint surfaces may be rough, smooth or slickensided.	From less than 1 metre, up to 10 metres.	Shrinkage on cooling (igneous rocks). Extension or shear (any rocks) due to tectonic stresses, or unloading.
	Sheared zone (1) (Fault)	Zone with roughly parallel almost planar boundaries, of rock substance cut by closely spaced (often <50 mm) joints and or cleavage surfaces. The surfaces are usually smooth or slickensided and curved, intersecting to divide the mass into lenticular or wedge-shaped blocks.	More than 10 metres, up to kilometres.	Faulting, during which small displacements occurred along slickensided surfaces distributed across the width of the zone.
	Crushed seam (1) (fault)	Seam with roughly parallel almost planar boundaries, composed of usually angular fragments of the host rock substance. The fragments may be of gravel, sand, silt or clay sizes, often mixtures of these. The finest fragments are often concentrated in thin layers or 'slivers' next to and parallel to the adjacent rock. Slickensides are common in the sliver zones and along the rock boundaries.	More than 10 metres, up to kilometres.	Faulting, during which relatively large displacements occurred within the zone.
	Soil infill seam	Seam of soil substance, usually with very distinct roughly parallel boundaries. Thin seams usually CH; thick near-surface seams sometimes GC or GC-CL (2).	Less than 10 metres, within near-surface mechanically weathered zone, or in rock disturbed by creep or landsliding.	Migration of soil into an open joint or cavity.
	Extremely weathered (or altered) seams	Seam of soil substance, often with gradational boundaries. The type of soil depends on the composition of the host rock.	Varies widely, but extremely weathered seams are usually within or near the near-surface weathered zone.	Weathering (or alteration) in place, of rock substance, usually next to or within a pre-existing defect.

NOTES
(1) In the geological sense.
(2) Group symbols as defined in the United States Bureau of Reclamation Earth Manual, 1963, Figure 3.1.
(3) A 'marker' feature, showing the nature of the displacements. Such features are observed only rarely.

Figure 2.1 Common defects in rock masses.

(indirectly, at least) from the stress histories. As a consequence of this, the defects invariably occur in more or less regular patterns, which can usually be distinguished at each site by the use of structural geology.

Further discussion of these defects and their significance in dam engineering is given in Sections 2.3.1 to 2.3.5. The characteristics of defects which occur in particular rock types or geological environments are discussed in Chapter 3.

2.3.1 Joints

Joints generally cause a rock mass to be less strong, less stiff and more permeable than an equivalent body of the rock substance. Where joints have been acting as conduits for ground waters, the rock next to them is often weakened by weathering (see Section 2.6). As joints are typically of limited extent, they do not usually present serious problems in embankment dam foundations but may be more significant in concrete gravity and arch dam foundations. Treatments for jointed rock usually include excavation, grouting and drainage (see Chapters 17 and 18).

2.3.2 Sheared and crushed zones (faults)

Regardless of whether its displacements have been normal, reverse or transcurrent, a fault may be a sheared zone, a crushed zone, or some combination of these two. In crushed zones the rock in the displacement zone has suffered brittle failure. In sheared zones the rock failure may have ranged from brittle to pseudo-ductile, where the shear displacements have been microscopic in size. Rarely, a single slickensided joint may prove to be a continuous and significant fault. This same fault could be a thin crushed seam (gravelly clay, CL-GC) where it passes through a shale bed, and a wider crushed zone (fine gravel GP) where it passes through a sandstone bed. Faults are important in dam foundations because they contain material which is usually

- of low strength and stiffness in shear,
- compressible and erodible and
- of large extent.

Some faults are more permeable than the average rock, or have open-jointed zones of high permeability next to them. Some clay or gouge filled fault zones are less permeable than the surrounding rock, and may act as low permeability barriers to groundwater movement. Often, the rock in and next to faults is weakened further by weathering or alteration.

Because of these characteristics, if present in foundations but undetected or inadequately treated or allowed for in designs, faults have the potential to seriously disrupt dam construction or operation, or even to cause or contribute to dam failure. A thin crushed seam contributed to the failure of Malpasset Dam in 1958. The role of this seam is discussed in Gosselin *et al.* (1960), Terzaghi (1962), Jaeger (1963), Londe (1967), Stapledon (1976) and James and Wood (1984).

The nature of treatment applied to a fault in a dam foundation will depend upon the designer's assessment of the capacity of the fault to adversely affect the behaviour

of the foundation and its interaction with the dam, during operation. Examples of treatments of faults are described in Chapter 17.

2.3.3 Soil infill seams (or just infill seams)

Soil infill seams are formed by soil which has gravitated, washed or squeezed into slots. Most of the slots would have been gaping joints in rock masses which have undergone either appreciable dilation due to stress relief (Figures 2.1 and 2.6) or disruption due to creep or landsliding (see Section 2.10). In soluble rocks (carbonates or evaporates) infill materials occur mainly in irregular cavities, but also as seams in slots resulting from widening of joints or thin beds, by solution (see Sections 3.7.1 and 3.8).

The infill materials are commonly clay (CH), and often contain roots or tubes remaining after the rotting of roots. Although individual infill seams are rarely of large lateral extent, they are likely to have low shear strength and to be compressible and erodible.

Figure 3.22b shows an example of infilled stress relief joints.

2.3.4 Extremely weathered (or altered) seams

Extremely weathered (or altered) rock is material which was once rock but has been converted by weathering (or alteration), in place, to soil material (see Sections 2.6.3, 2.7, 2.8.1 and Tables 2.3 and 2.6). The extremely weathered seam (a) in Figure 2.1 has formed by the weathering of a thin bed of shale next to its boundary with a sandstone bed. Some such seams in this type of situation show slickensided surfaces, which can indicate either:

- The seam was, or has formed next to, a bedding-surface fault (Figure 3.19), or
- It is simply a weathered bed within which some displacement has occurred since it became weathered.

Extremely weathered seam (b) has developed along or next to a joint.

Extremely weathered (or altered) seams can be of large or small extent. Their shear strengths, shear stiffnesses, compressibilities and erodibilities, will depend largely on the compositions and fabrics of the parent rocks. Weathered seams normally occur within or not far below the near-surface weathered zone (see Section 2.6.3), but altered seams can occur at any depth.

2.3.5 The importance of using the above terms to describe defects in rock

Use of the terms defined in Figure 2.1 and discussed above can enable site investigators to compile geotechnical models in an efficient and economical manner, due to the following:

- Individually, they are based on geological understanding, and used together they allow the history of the site to be understood.

- They form more reliable bases for correlation between data points (exposures in boreholes or pits) than purely engineering descriptions of the materials which form them.
- Some of their engineering properties, and their probable shapes and extents, can be predicted from the defect names.
- They can provide evidence that a rock mass has been mechanically loosened, e.g. by mechanical weathering, landsliding, or volcanic explosion.

ISRM (1975) use the general terms "infilling" or "filling" to describe any materials (usually with soil properties) which form, or occur within the boundaries of, defects (or discontinuities) in rock masses. This generalization may have been a useful shorthand for some engineers or those involved in mathematical modelling, but its use in engineering for dams is not recommended because:

- It provides none of the above-listed benefits of the terms on Figure 2.1 and
- It can be confused with the "infill" or "soil infill" term of Figure 2.1.

The authors recommend that if a seam or zone cannot be confidently identified in terms of Figure 2.1, then the material in it should be described as "seam material" rather than "infilling".

2.4 DEFECTS IN SOIL MASSES

Soils formed by the extreme weathering of rocks can inherit any of the fabrics described in Section 2.2. Also they may contain remnant joints, crushed zones or sheared zones. The contrast between the strengths of these remnant defects and that of the soil (extremely weathered rock substance) is usually lower than in a comparable mass of less weathered or fresh rock.

Soils of sedimentary origin can contain individual thin beds which behave effectively as defects. For example, a thin silty or sandy bed in a clay deposit can represent a leakage path with permeability several orders higher than that of the adjacent soil. Also, a thin clay bed in a sandy soil can form a local barrier to water flow and a surface of low shear strength.

Soils of any origin type can contain defects of the following types:

- Fissures (or joints) – near-planar surfaces, usually polished or slickensided, formed in clayey soils by (usually) small shear displacements associated with swelling.
- Tension cracks (or joints) – near-planar surfaces formed by extension due to shrinkage, subsidence, landsliding or tectonic displacements.
- Remoulded zones – zones of remoulded and softened soil, often containing fissures, formed by appreciable shear displacements due to subsidence, landsliding or tectonics.
- Tubular or irregular holes – formed by burrowing animals, rotted roots, or erosion by seeping or flowing water.
- Infilled features – previously open joints or holes, wholly or partly filled with younger soil.

All of these defect types generally lower the strength and stiffness, and increase the permeability of soil masses (Stapledon, 1971; Walker *et al.*, 1987; MacGregor *et al.*, 1990; Moon, 1992).

2.5 STRESSES IN ROCK MASSES

Measurements of *in situ* rock stresses at shallow depths in many geological environments throughout the world have shown horizontal stresses which are generally higher than can be explained theoretically from the weight of the present overburden. Hast (1967) published the results of a large number of stress measurements in relatively intact granitic and metamorphic rocks in Scandinavia. Hast showed that at all Scandinavian sites, the major and intermediate principal stresses were horizontal, and that the sum of these two stresses (Line A on Figure 2.2) increased linearly with depth. Also plotted on Figure 2.2 are the mean horizontal stress (Line B) and the theoretical vertical stress due to overburden weight (Line C). It can be seen that within about 50 m of the ground surface the mean horizontal stress appears likely to be more than ten times the vertical stress.

Brown & Hoek (1978) compiled Figure 2.3 and Figure 2.4 from stress measurements at mining and civil engineering sites throughout the world. Figure 2.3 shows that the vertical components of the measured stresses are in fair agreement with the calculated vertical stress due to overburden. Figure 2.4 shows the variation with depth of the ratio of the mean horizontal stress to the vertical stress. It can be seen that

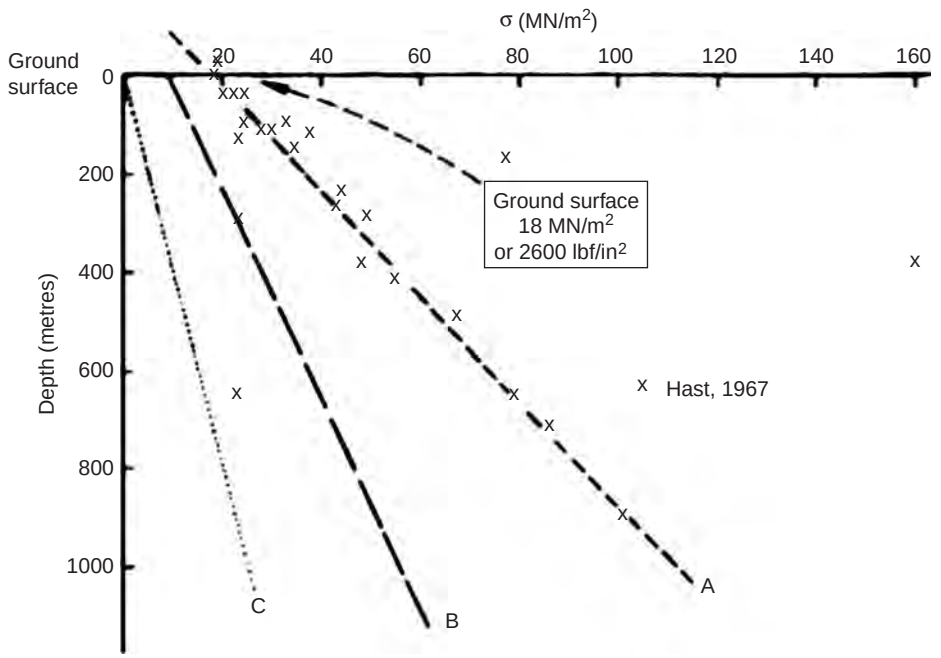


Figure 2.2 Plot of horizontal stresses against depth in Scandinavian mines (based on Hast, 1967).

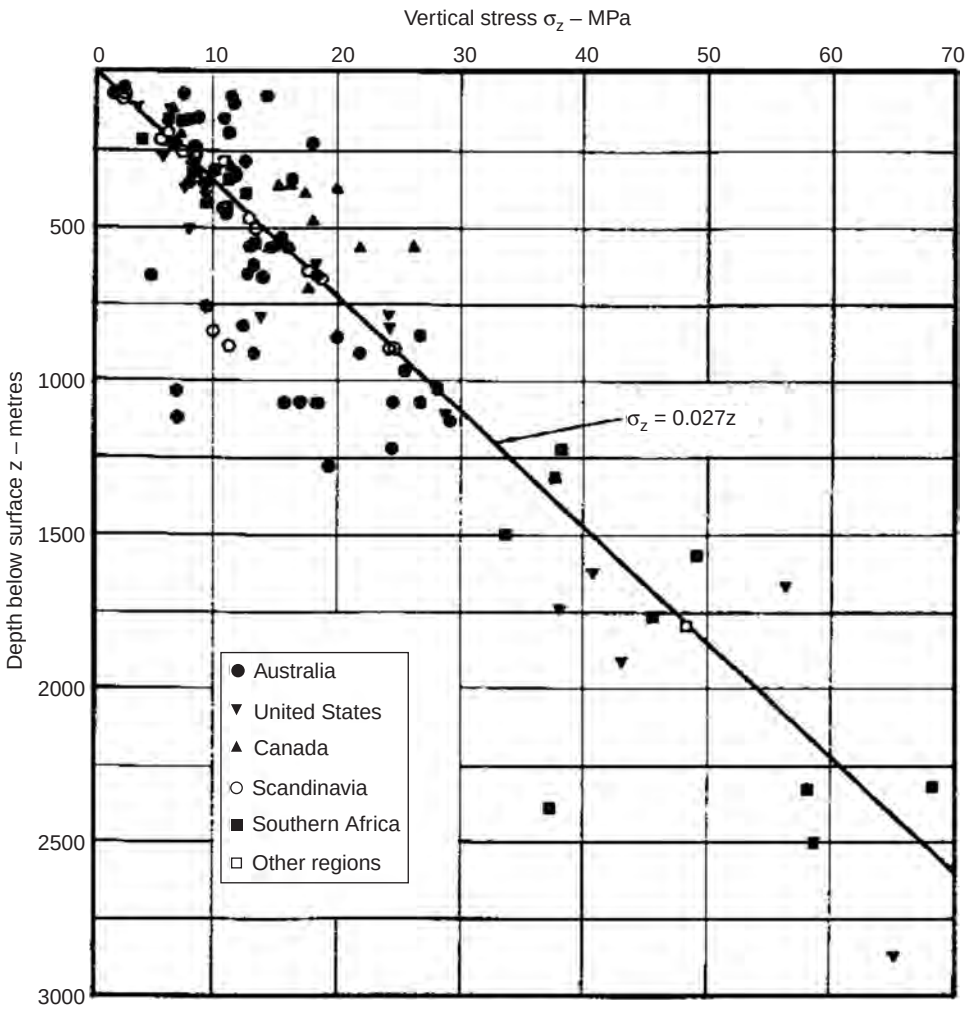


Figure 2.3 Plot of vertical stresses against depth below ground surface (from Brown and Hoek, 1978, by permission of Pergamon Press).

horizontal stresses are generally significantly greater than vertical stresses, at depths of less than 500 m and that at greater depths the stresses tend to equalize.

2.5.1 Probable source of high horizontal stresses

The prime source of high horizontal stresses is believed to be tectonic forces, i.e. the forces which drive and resist the motion of the earth's crustal plates. Evidence for this comes mainly from depths of more than a kilometre below the surface and is provided by analyses of spalling (breakouts) and tensile fractures in the rock around deep wells drilled for petroleum.

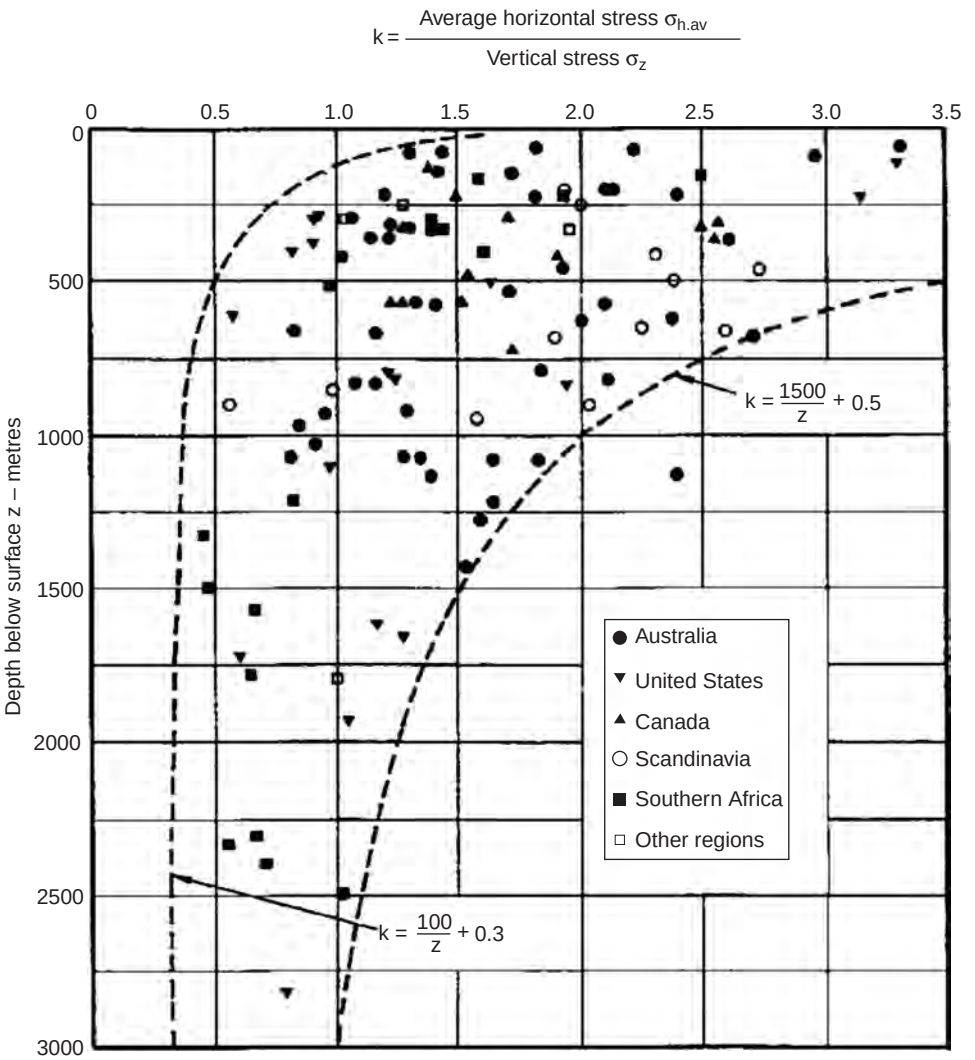


Figure 2.4 Variation in ratio of average horizontal stress to vertical stress with depth below surface (from Brown and Hoek, 1978, by permission of Pergamon Press).

The horizontal stress fields inferred at these great depths have been related generally to plate motion, and a world stress map has been produced (Zoback, 1992). There is also a stress map covering Australia and Papua New Guinea (Hillis *et al.*, 1998).

At the much shallower depths relevant to dam projects, the local stress fields often cannot be related directly to the regional tectonic stress field at depth. The differences are probably caused by local effects such as variations in topography or structure of the bedrock.

It is possible also in some situations that the near-surface stresses result from strain energy which has been locked into rocks during their formation, often, but not necessarily, at great depth. This may have occurred in igneous rocks during their solidification, and in sedimentary and metamorphic rocks, during compaction, cementation or recrystallization; Emery (1963), Savage (1978), Nicholls (1980), Brown and Windsor (1990) and Brady and Brown (1985). At a microscopic level the strain energy is considered to be locked into mineral crystals or grains by cementation and interlocking. An analogous model would be compressed springs embedded in plastic. As the vertical load on high stressed rock is slowly lowered by erosion, vertical stresses are relieved progressively by upward expansion. However, because the rock remains confined laterally, the horizontal stresses decrease in accord with Poisson's Ratio, i.e. at about one-third of the rate of the vertical stresses. This results in the recorded near-surface imbalance.

2.5.2 Stress relief effects in natural rock exposures

Field evidence of the existence of the high horizontal stresses at shallow depths is seen most clearly in areas of massive igneous rocks which contain very few tectonically induced fractures. Most of Hast's measurements plotted on Figure 2.2 were made in such rocks. These rocks usually contain "sheet joints" near-parallel to the ground surface, as shown in Figure 2.5. The sheet joints show rough, irregular, plumose surfaces indicating that the rock failed in tension, by buckling or spalling, the tension being induced by the high horizontal compressive stress. Some sheet joints show slickensides near their extremities indicating local shear failure which should be expected here (Figure 2.5). The spacing of sheet joints is often 0.3 m to 1 m near the ground surface and becomes progressively wider with depth.

From all of these characteristics, sheet joints are considered to be stress-relief features. The linear decrease in horizontal stresses as the ground surface is approached (Lines A and B, Figure 2.2) is due to the progressive relief of horizontal stresses partly by buckling and spalling, as the overlying rock load is removed by erosion.

Holzhausen (1989) provides a comprehensive account of the characteristics and origin of sheet joints.

In fractured rock masses, i.e. those which are already weakened by defects of tectonic origin, the effects of horizontal stress relief are not so obvious but are always present, as opening up of the existing defects, as shown in Figure 2.6b. The destressing effects usually extend to greater depths in jointed rock than in massive rock. However, in most geological situations it can be assumed that beyond the near-surface effects, the tectonically induced joints will become progressively tighter (less open) with greater depth (Snow, 1970).

2.5.3 Effects in claystones and shales

Claystones and shales (i.e. "mudrocks") are formed mainly by consolidation of clay-rich sediments, but may be strengthened further by partial recrystallization and cementation. Such rocks have much higher porosities than igneous and metamorphic rocks. They usually show expansion, spalling and fretting on unloading and exposure. It is believed that these effects are due partly to the release of stored strain energy

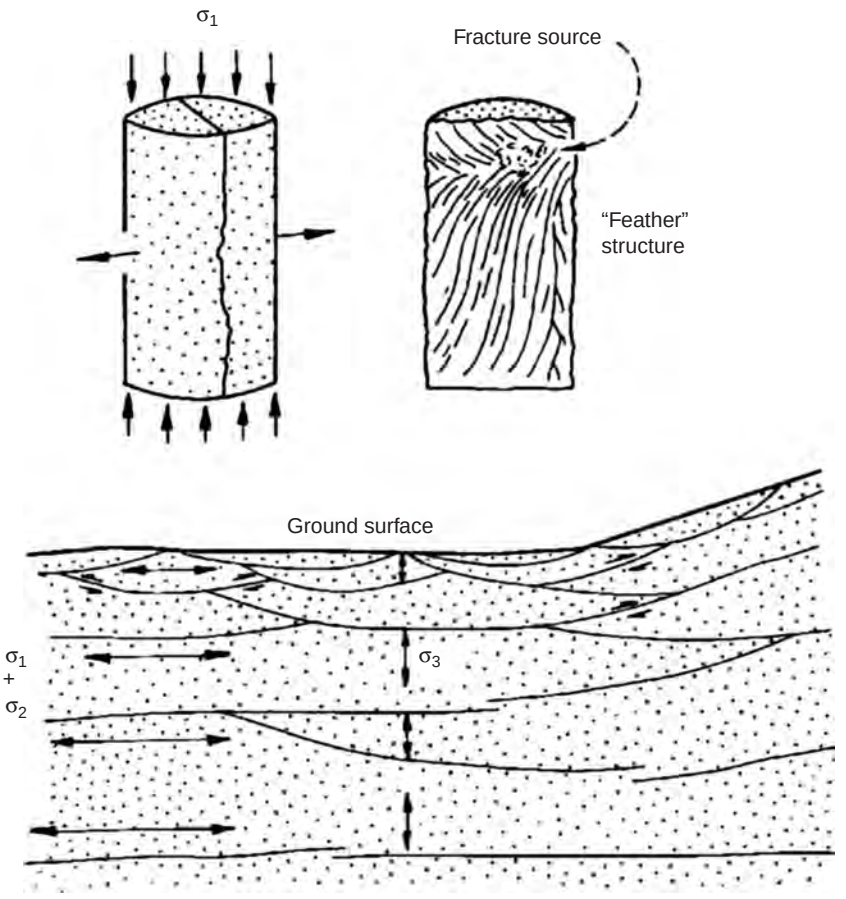


Figure 2.5 Sheet-joints, formed by induced tensile failure (as in an unconfined compression test with zero friction at the plattens).

(Bjerrum, 1967) but also to the absorption of water and subsequent swelling of the clays (See Section 2.9.1 and Section 3.5.1).

2.5.4 Special effects in valleys

Gentle anticlines, in some cases with associated thrust faults as shown in Figure 2.7, have been recorded across many river valleys cutting through near-horizontal sedimentary rocks of moderate to low strength. The phenomenon is referred to as "valley bulging" or "valley rebound" and has been described by Zaruba (1956), Simmons (1966), Ferguson (1967), Patton and Hendron (1972), Matheson and Thompson (1973), Horswill and Horton (1976), McNally (1981) and Hutchinson (1988).

Most of the features shown on Figure 2.7 have clearly developed as a result of buckling and shear failure under high horizontal compressive stresses. The stresses

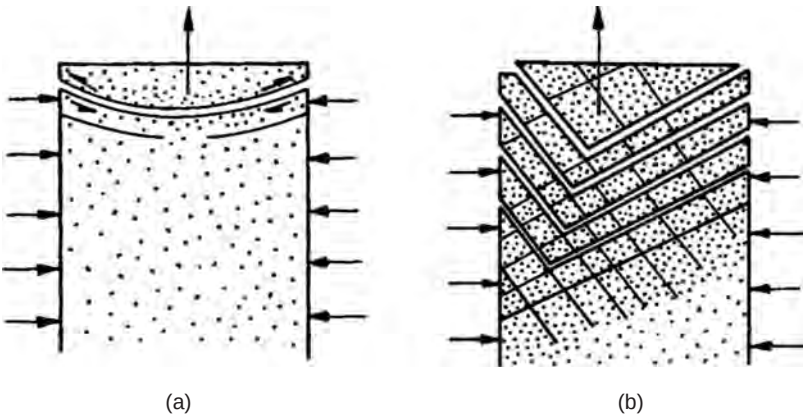


Figure 2.6 Effects of destressing in (a) intact rock, and (b) jointed rock.

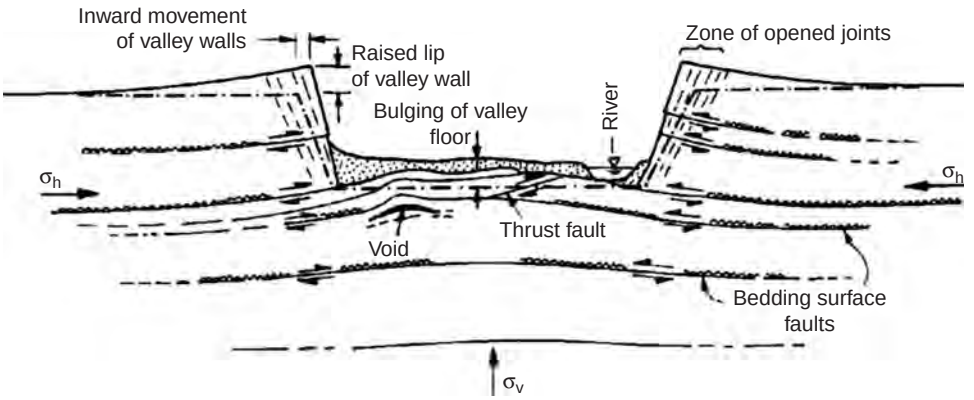
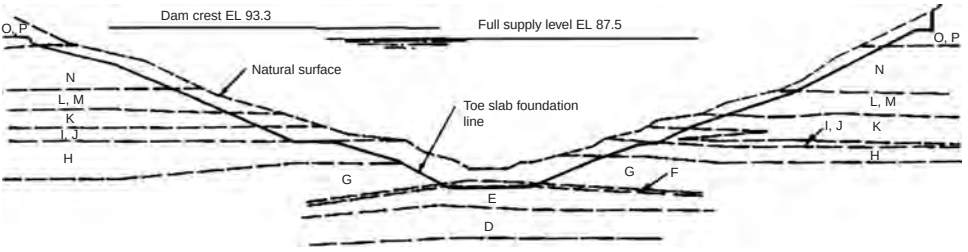


Figure 2.7 Complex valley structures related to stress release in weak, flat-lying rocks (based on Patton & Hendron, 1972).

were concentrated beneath the valley floor as a result of load transfer as the excavation of the valley removed lateral support from the rock layers above the floor and vertical load from the rock beneath the floor. The steeply-dipping joints next to the cliff faces probably opened up due to expansion of the rock layers under the influence of horizontal stresses both across and parallel to the valley.

All of the effects shown on Figure 2.7 were present at the site for Mangrove Creek Dam near Gosford, New South Wales. This 80 m high concrete faced rockfill dam is located in a valley 200 m to 300 m deep, cut through an interbedded sequence of sandstones, silt-stones and claystones (Figure 2.8). Away from the river the rock layers in the valley sides generally show joints at wide to very wide spacing.

Near and beneath the river bed there is a broad, gentle, "valley bulge" as shown on Figure 2.8. Although the shape of this feature is not very pronounced, the rock



Abbreviation	Material
D	Interlaminated siltstone, claystone and minor sandstone.
E	Sandstone
F	Siltstone, claystone
G	Interbedded interlaminated sandstone, siltstone and claystone
H	Coarse grained sandstone

Abbreviation	Material
I, J	Laminated siltstone
K	Fine medium grained sandstone
L, M	Interlaminated siltstone and sandstone
N	Sandstone
O, P	Sandstone with interbedded siltstone

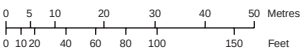


Figure 2.8 Cross-section along the grout cap foundation at Mangrove Creek Dam (MacKenzie & McDonald, 1985).



Figure 2.9 Upper surface of Unit E sandstone at Mangrove Creek Dam valley bulge, showing joints open as much as 100 mm, clay-filled in part.

within it was intensely disrupted to 15 m below the river bed. The sandstone unit E (Figure 2.9) showed extension joints which were open or clay-filled, to a maximum of 100 mm, and the underlying unit D (interbedded) contained zones of crushed rock and clay up to 600 mm thick, apparently produced by overthrust faulting (Figure 2.10). This unit also contained gaping joints parallel to bedding.



Figure 2.10 Crushed zone in Unit D (Interbedded siltstone and claystone) formed by over-thrusting. Photograph courtesy G. McNally.

To provide a stable foundation for the grout cap and to prevent possible erosion of the clay filling from joints, cable anchors were installed and a concrete diaphragm wall 30 m long and 15 m deep was constructed beneath the valley floor (MacKenzie and McDonald 1985).

Although usually not as pronounced as in Figure 2.8 to Figure 2.10, destressing effects are invariably found at and near the floors of valleys in strong to extremely strong rocks, in areas where high horizontal stresses are known to exist. The effects include sheet joints and gaping or soil-infilled joints of tectonic origin. These joints usually occur both in the valley walls and beneath the floor. It is clear from Figure 2.11a that large stress concentrations are likely to occur in rock beneath the bottom of deep, youthful gorges, and so such destressing effects, even if not visible, are likely to be present, as shown in Figure 2.11b.

2.5.5 Rock movements in excavations

Spalling, buckling upwards, and the formation of new sheet joints are relatively common occurrences in near-surface excavations in very high strength rocks in North America, Scandinavia and Australia. Emery (1963) reports that an anticlinal fold about 100 m long with crest elevation of almost 5 m was formed overnight in a limestone quarry in Kingston, Ontario. Lee *et al.* (1979) describe rock bursts in shallow excavations in granite and gneiss in Maine, U.S.A. Ward (1972) describes a 30 m deep excavation in gneiss at New York City, where inward movement of the walls caused rock bolt failures.

Bowling and Woodward (1979) describe "rock bursts" and the formation of new sheet joints at Copeton Dam in New South Wales. Relatively minor spillway discharges of up to 460 m^3/s caused erosion of a channel about 20 m deep in fresh, massive granite.

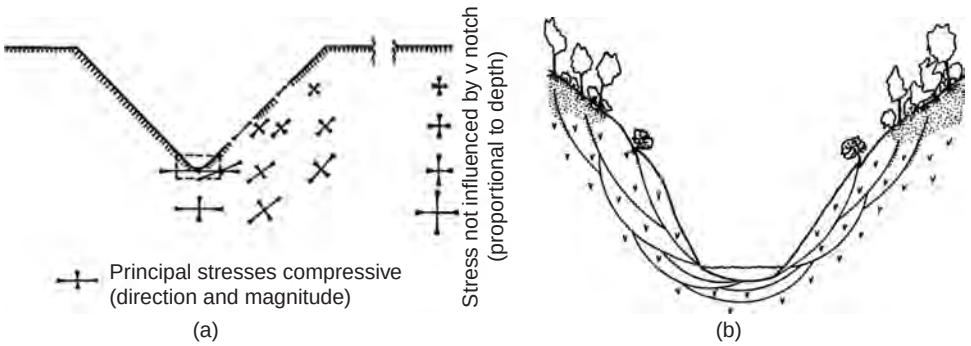


Figure 2.11 High stresses developed in valley floor and lower sides, and resulting hypothetical pattern of sheet joints in strong rock without "tectonic" joints (model results from Alexander, 1960).

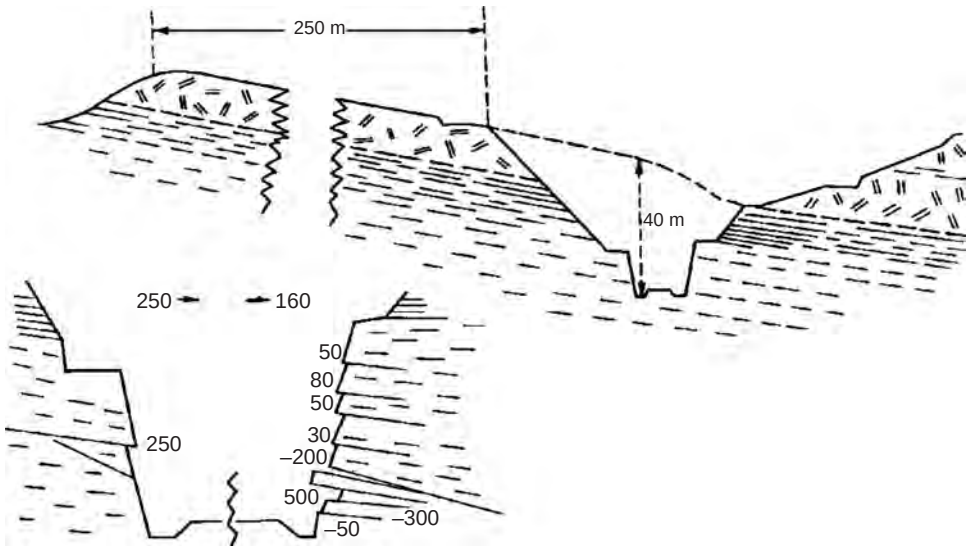


Figure 2.12 Convergence indicated by traces of presplit holes, near base of railway cutting in fresh, strong siltstone.

The granite apparently failed progressively by buckling upwards. As each slab of rock was carried away a new sheet joint formed, by buckling. Isolated rock bursts continued to occur in the channel floor for more than 6 months after the floods. The channel was eventually stabilized by rock anchors and dental concrete.

Inward movement or "convergence" of the walls of relatively shallow excavations in rock is another common destressing effect. Figure 2.12 shows part of a railway cutting 40 m deep and about 500 m long near Paraburdoo, Western Australia. After excavation of the lowest 15 m of the cutting, by presplitting and trench-blasting methods, a maximum possible convergence of 410 mm was indicated by displacement of traces of presplit holes. Although part of this convergence was undoubtedly caused by

the development of blast-initiated cracks behind the rock faces, it is believed that much of the 410 mm was caused by destressing of the gently dipping siltstone. This belief is based on many other similar situations where the authors have observed inward movements to continue for several days after completion of excavations. Similar observations have been reported by Wilson (1970).

2.6 WEATHERING OF ROCKS

Broadly speaking, weathering of a rock is its response to the change from the pressure, temperature, moisture and chemical environments in which it was formed, to its new environment at and near the ground surface. Weathering processes are of two fairly distinct types, namely mechanical and chemical.

2.6.1 Mechanical weathering

Mechanical weathering includes all of the near-surface physical processes which break rock masses down to progressively smaller rigid blocks or fragments and cause those blocks to separate. Mechanical weathering generally precedes chemical weathering. It renders the rock mass more permeable and facilitates access for groundwater to large surface areas of rock substance.

Destressing, in particular the formation of sheet joints and the opening up of existing "tectonic" joints near the ground surface, is the primary and generally the most significant mechanical weathering process (See Figures 2.6, 2.7, 2.9 and 2.11b). The other processes, in order of their (generally) decreasing significance, are as follows:

- gravitational creep (e.g. of slabs and wedges, and toppling)
- joint water thrusting and uplifting during extreme rainfall events
- earthquake induced displacements
- growth of tree roots in joints
- expansion of clays in joints
- freezing of water in joints
- extreme temperature changes causing differential expansion and contraction of exposed rock faces.

Patton and Hendron (1972) suggested possible mechanical weathering effects beneath the floor and lower sides of valleys in groundwater discharge areas. This type of situation (Figure 2.13) may be relatively rare, but if it occurs the hydraulic uplift and thrust effects shown are clearly possible.

2.6.2 Chemical decomposition

The term chemical decomposition as used here includes all of the chemical (and to a minor extent physical) processes which cause mineral changes resulting generally in weakening of rock substances, so that eventually they assume soil properties.

Throughout this book, the products of chemical decomposition are described using the terms defined in Section 2.8.1, Tables 2.3 to 2.6.

Chemical decomposition of rocks can be caused by either near-surface (weathering) processes or deep-seated (alteration) processes. Recognition of this distinction

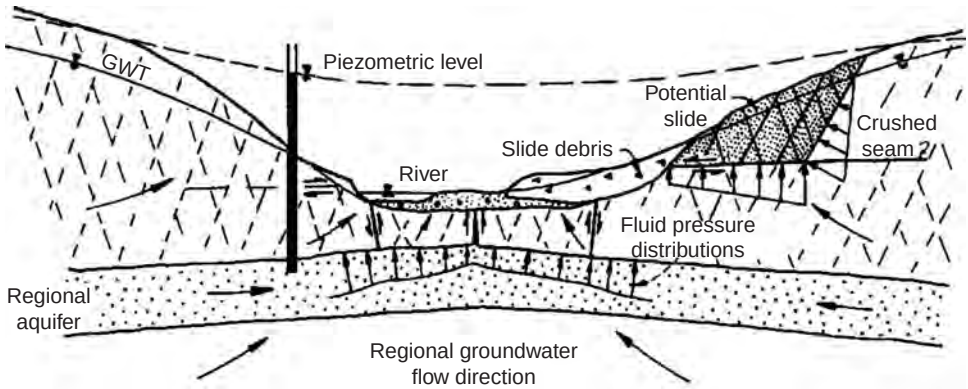


Figure 2.13 Possible effects of high fluid pressures on valleys in groundwater discharge areas (from Patton and Hendron, 1972).

is important in civil and mining engineering because the nature and distribution of weathered materials are generally different from those of altered materials (Figure 2.14).

2.6.3 Chemical weathering

Chemical weathering is caused mainly by circulating groundwater which gains access to low-porosity rock substances via cleavage micro-cracks, open joints and fractures associated with faults. In the case of more porous rocks, e.g. some sandstones and limestones, groundwater can also enter through intergranular pores. Most chemical weathering occurs at extremely slow rates, such that the changes to the strength of high strength, non-porous rocks (e.g. granite) are likely to be insignificant during the operating life of most civil engineering projects. There are however some minerals and rocks which decompose, weaken or disintegrate within a few months or years of exposure. These effects will be discussed separately under Section 2.9, Rapid weathering.

Chemical weathering involves the more or less continuous operation of all or most of the following:

1. Chemical reactions between the minerals in the rock, and water, oxygen, carbon dioxide and organic acids. These reactions cause decomposition of the minerals to form new products, some of which are soluble.
2. Removal of the soluble decomposition products by leaching.
3. Development of microcracks in some rocks, probably due to some decomposition products having larger volumes than the original minerals or to destressing or capillary or osmotic suction effects.
4. Deposition of some decomposition products, in pores or microcracks.

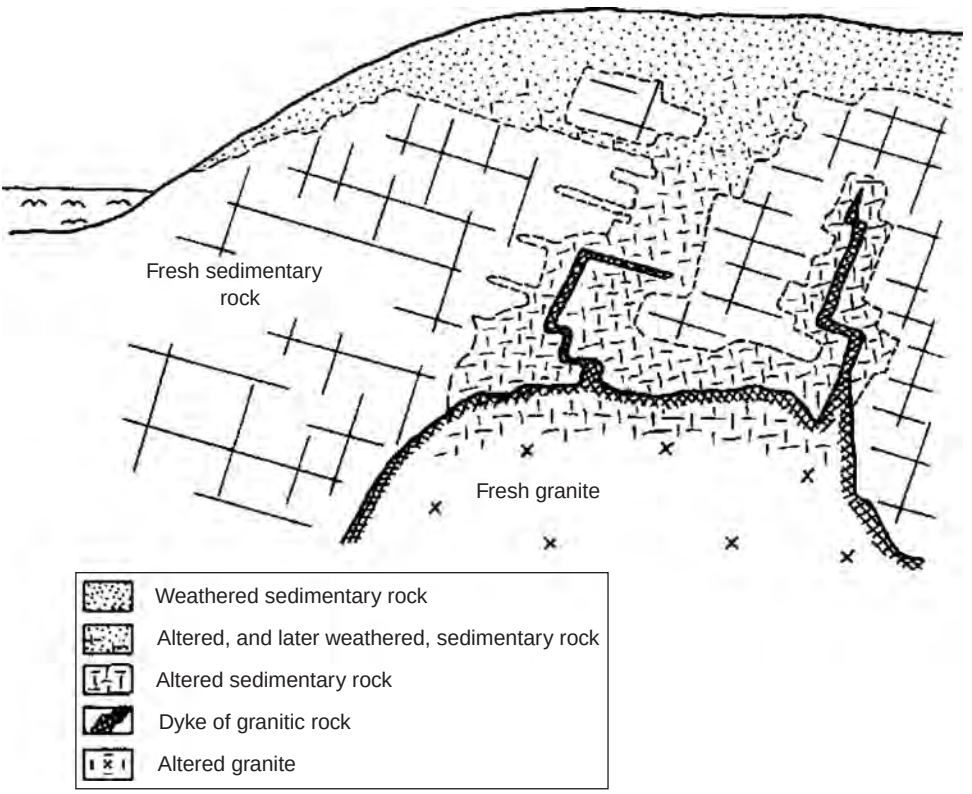


Figure 2.14 Diagrammatic cross section showing hydrothermally altered zone near a granite intrusion, with the uppermost part of the altered zone being weathered subsequently.

Processes 3 and 4 were illustrated by Dixon (1969) who made microscopic studies of granite and schist in a range of weathered conditions. He found that in slightly weathered samples, i.e. rocks which showed only slight discolouration in hand specimen, the first weathering effects visible microscopically were slight discolouration of the felspars and mica minerals, and the presence of many microcracks, some open and others filled with an opaque mineral assumed to be limonite or clay (Figure 2.15).

Baynes and Dearman (1978a) describe microfabric changes which occur in granites during various stages of weathering, using a scanning electron microscope. In Baynes and Dearman (1978b) they relate changes in engineering properties to the microfabric changes.

The chemical reactions involved in chemical weathering include carbonation, hydrolysis, solution, oxidation and reduction. As most cannot be actually observed, details of the reactions, as published by various workers, are in part speculative. Useful accounts are given by Selby (1993), Price (1995) and the Geological Society Engineering Group (1995).

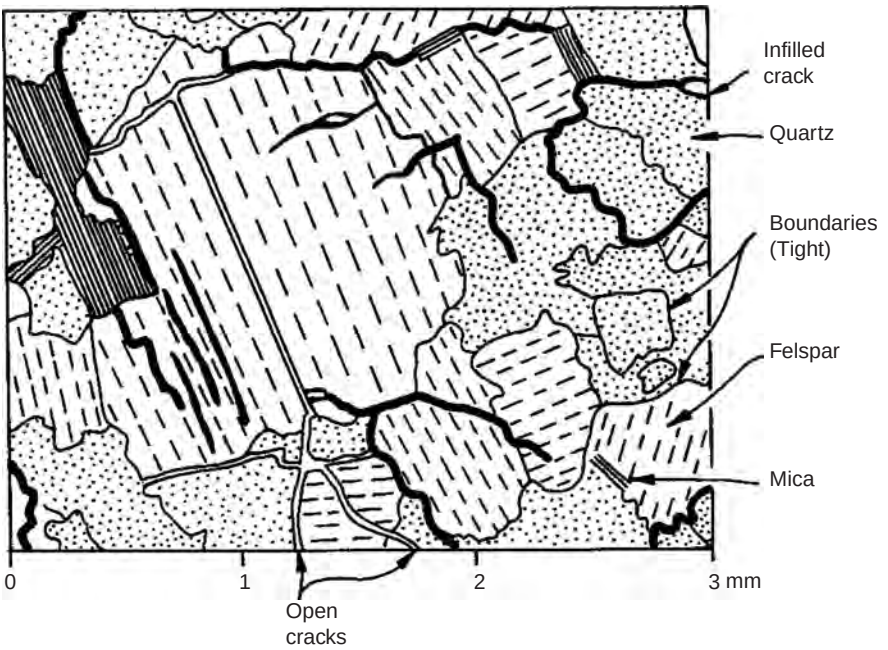


Figure 2.15 Microscopic view of the structure of slightly weathered granite (Dixon, 1969).

Table 2.1 Susceptibility of igneous rock-forming minerals to weathering.

Temperature of formation	*Susceptibility to weathering*	*Mineral*	*Common igneous rock types*
Highest	Highest	Olivine	
			Basalt, dolerite, gabbro
		Calcic felspar	
		Augite	Andesite, diorite
		Hornblende	
		Sodic felspar	
		Biotite	
			Rhyolite, granite
		Muscovite	
Lowest	Lowest	Quartz	

2.6.3.1 *Susceptibility of common minerals to chemical weathering*

As would be expected, the susceptibility to weathering (or the "weatherability") of minerals in igneous rocks varies in accordance with the temperatures at which they were formed. This is illustrated in Table 2.1. With the exception of quartz, the most stable mineral, all of the others on Table 2.1 weather eventually to clay minerals. Quartz is slightly soluble in water. It is almost unaffected by weathering except under tropical conditions when it is readily dissolved (in a geological time-frame).

Table 2.2 Susceptibility of other common minerals to weathering.

Group	*Mineral*	*Effects of weathering*
Carbonates	Calcite	Readily soluble in acidic waters
	Dolomite	Soluble in acidic waters
Evaporites	Gypsum	Highly soluble
	Anhydrite	Highly soluble
	Halite (common salt)	Highly soluble
Sulphides	Pyrite and various other pyritic minerals	Weather readily to form sulphates, sulphuric acid and limonite
Clay minerals	Chlorite	Weathers readily to other clay minerals and limonite
	Vermiculite	Weathers to kaolinite or montmorillonite*
	Illite	Weathers to kaolinite or montmorillonite*
	Montmorillonite	Weathers to kaolinite
	Kaolinite	Stable**
Oxides	Haematite	Weathers to limonite
	Ilmenite	Stable
	Limonite	Stable

*These minerals expand and contract with wetting and drying and this can cause large disruptive forces and disintegration of some rocks.
**Softens on wetting.

The susceptibility of other common minerals to weathering is indicated in Table 2.2. The minerals in the carbonate and evaporite groups (shown in this table) may occur in any of three different ways, namely:

- as rocks, e.g. calcite as limestone, or dolomite as dolomite (rock) or dolostone or
- as cements in sedimentary rocks, e.g. calcite as cement in sandstone composed mainly of quartz grains or
- as veins or joint fillings or coatings in any rock mass.

2.6.3.2 *Susceptibility of rock substances to chemical weathering*

The susceptibility of a rock substance to weathering depends upon the following:

- the susceptibility to weathering of its component minerals
- the nature of its fabric, i.e. the degree of interlock and/or cementation of the mineral grains
- its porosity and permeability.

For example, both rhyolite and granite contain sodic felspar, micas and quartz, but in rhyolite the crystals are much more fine grained and more tightly interlocked than in granite. Hence granite is much more susceptible to weathering than rhyolite (Figure 2.16).

Also, a dense, non-porous limestone comprising almost 100% calcite is likely to be less susceptible to weathering (solution) than a porous sandstone comprising 80% quartz grains which are durable but cemented by calcite.

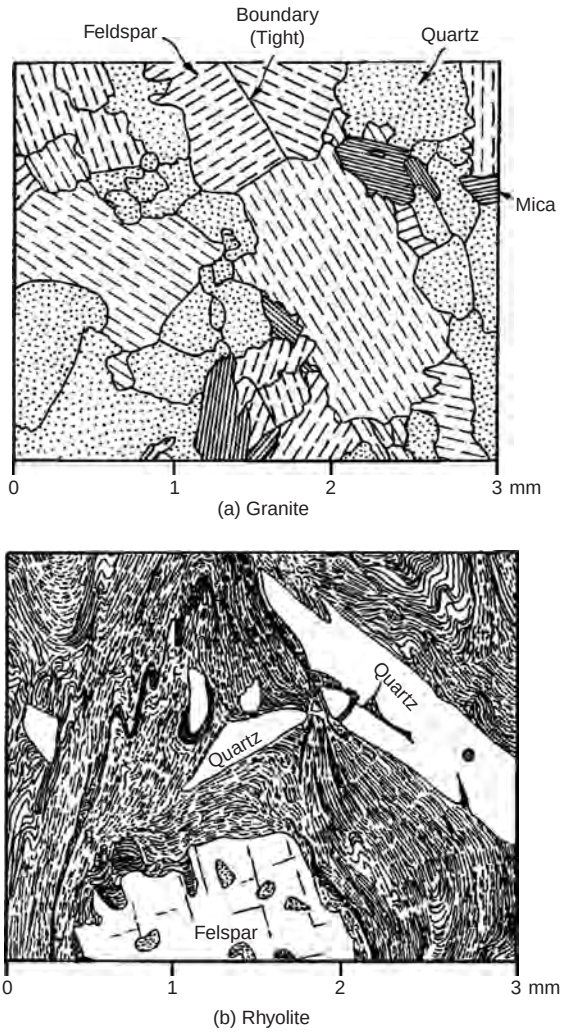


Figure 2.16 Microscopic views of the structure of (a) granite and (b) rhyolite.

2.6.4 Weathered rock profiles and their development

The following are the main factors which contribute to the development of weathered profiles:

- climate and vegetation
- rock substance types
- defect types and patterns
- erosion
- time

– topography
– groundwater.

The influence of each of these factors is discussed in the sections which follow. Although the factors are discussed separately or in pairs, they usually all interact and influence the development of weathered profiles. The terms used to describe weathered rocks are defined in Section 2.8.

2.6.4.1 *Climate and vegetation*

Climate is the dominant factor. At one extreme, in a desert situation, chemical weathering effects are usually almost negligible. Mechanical weathering effects may include opening up of joints due to destressing and some fragmentation of exposed rock surfaces due to extreme temperature changes. At the other extreme, under hot, humid (e.g. tropical or subtropical) conditions chemical weathering proceeds relatively rapidly, due to the ready availability of water containing oxygen, carbon dioxide and organic acids derived from the vegetation. Regardless of the composition and structure of the parent rock mass, the near- surface weathered profile is usually of the lateritic type, the upper part of which may consist almost entirely of oxides of iron and aluminium. Figure 3.40 shows a typical near-surface profile this type and the processes believed to be involved in its development. It must be appreciated that the weathered profile at a particular site may not have been developed under the present climatic conditions. Throughout some arid areas of Australia and South Africa there are deep weathered profiles which developed under tropical or semi tropical conditions, largely during Tertiary time.

2.6.4.2 *Rock substance types and defect types and pattern*

Many fresh rock masses are relatively complex in their composition and structure. They may contain several different rock types, with widely differing substance strengths and susceptibilities to chemical weathering and the rocks may be folded and intersected by defects such as joints and faults. The material in the fault zones may be crushed rock which has essentially "soil" properties in the fresh (unweathered) state. Chemical weathering generally proceeds from the joints and faults, which act as groundwater conduits. Because of this, the distribution of intensely weathered rock is usually governed as much or more by the pattern of occurrence of these defects, than by the depth below the ground surface.

Figures 2.17 to 2.24 show a range of weathered rock profiles, illustrating these effects.

Figure 2.17 is a profile developed under tropical conditions in a gneiss rock mass with very simple structure – sheet joints parallel to the original (sloping) ground surface. The uppermost 13 m comprises a laterite soil profile overlying extremely weathered gneiss. The extremely weathered gneiss is a gravelly clay (CL-GC). Its upper boundary with the pallid horizon of the laterite profile is gradational, but its lower boundary with slightly weathered gneiss is sharp, coinciding with Sheet Joint (1). It is likely that several other sheet joints were present initially at shallower depth, but all trace of them has been obscured by the extreme weathering or by disturbance of the exposed surface of the cut. Between Sheet Joints (1) and (2) the gneiss is slightly weathered except for a narrow extremely weathered zone which surrounds Sheet Joint (2). Below this zone the

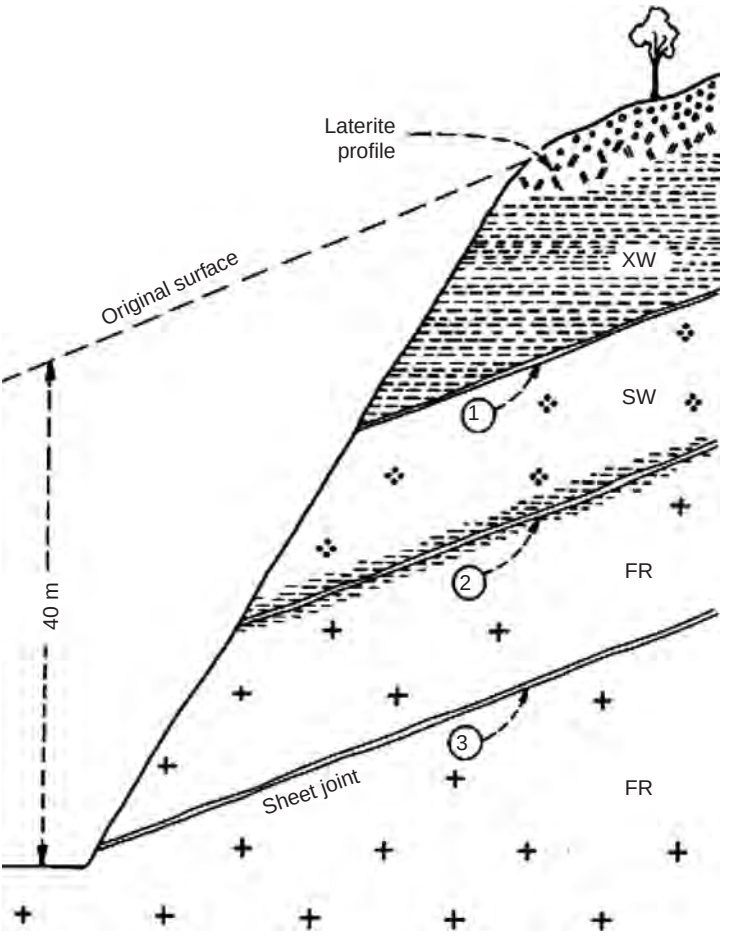


Figure 2.17 Weathered profile developed previously on sheet-jointed gneiss, Darling Range, Western Australia (Gordon, 1966).

gneiss is fresh. It is clear from this simple profile that chemical weathering proceeded both from the ground surface and from the sheet joints.

Figure 2.18 shows a more complex profile developed under tropical conditions in the same region as the profile in Figure 2.17. The profile steps downwards (i.e. deepens) to the east because of the presence of three different types of granite which are progressively less siliceous and hence are more susceptible to chemical weathering. The dolerite dyke has a high resistance to weathering and so forms a prominent ridge in the upper surface of the dominantly fresh rock zone. The contact between the dyke and quartz feldspar granite is sheared and weathering of the sheared rock has resulted in a deep, narrow slot of highly to extremely weathered rock. A local depression in the fresh rock zone occurs also along a sheared zone near the eastern side.

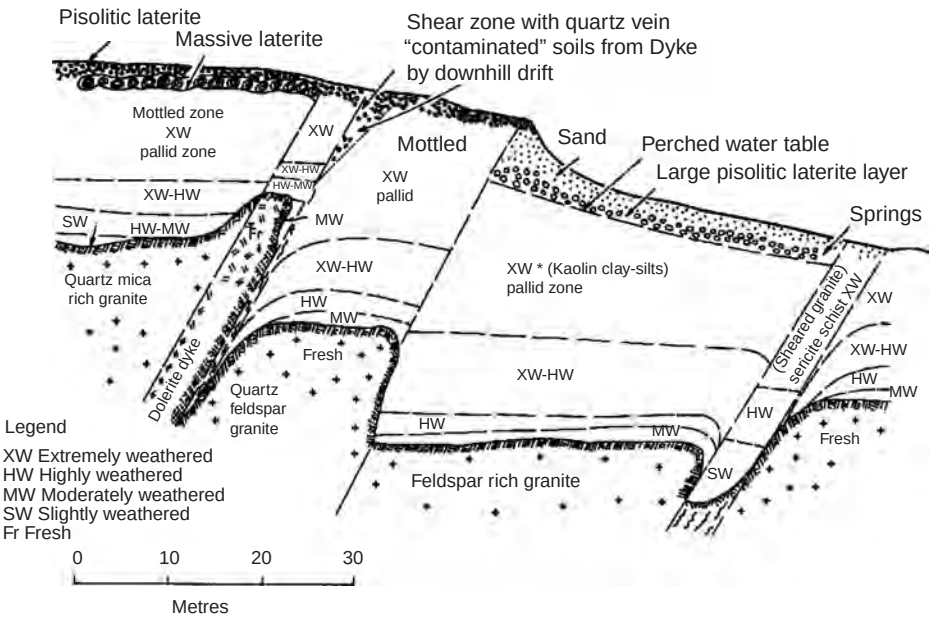


Figure 2.18 Diagrammatic cross section showing weathered profile controlled partly by rock type and partly by sheared zones, Darling Ranges, Western Australia (Gordon, 1984).

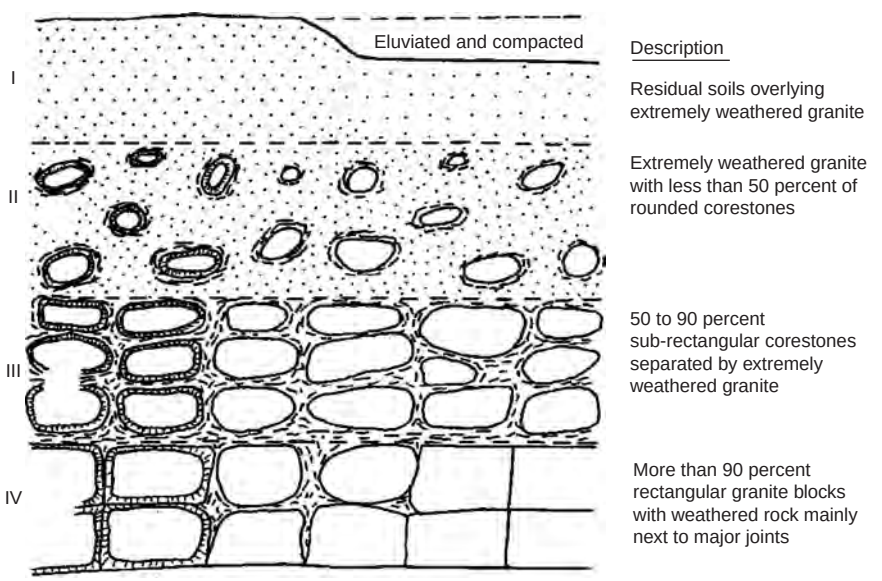

Eluviated and compacted
I
II
III
IV
Description
Residual soils overlying extremely weathered granite
Extremely weathered granite with less than 50 percent of rounded corestones
50 to 90 percent sub-rectangular corestones separated by extremely weathered granite
More than 90 percent rectangular granite blocks with weathered rock mainly next to major joints

Figure 2.19 Idealised weathered profile in granitic rocks (Ruxton and Berry, 1957).



Figure 2.20 Granitic boulders or corestones showing spheroidal weathering effects. Photo courtesy of Dr. R. Twidale.

Figure 2.19 shows an idealised weathered profile through granitic rocks in Hong Kong described by Ruxton and Berry (1957). Broadly similar profiles are found in many other areas underlain by granitic and other igneous rocks, and this profile has been widely accepted as typical for such rocks. It is clear that the main controls on the



Figure 2.21 Granite corestones and outcrops below granitic soils on Granite Island, Victor Harbor, South Australia.

distribution of weathered materials have been the depth below the ground surface and the pattern of the joints in the rock mass.

The corestones shown on Figure 2.19 usually display spheroidal weathering effects, that is, they comprise fresh or slightly weathered rock surrounded by concentric shells of rock which becomes progressively more weathered away from the core, as shown on Figure 2.20.

The cracks which isolate the shells are like small scale sheet joints and are probably caused by relief of residual stresses and other stresses set up by capillary, osmotic and other chemical weathering processes.

Figure 2.21 shows what initially appears to be an excellent example of the idealised weathered granitic profile of Ruxton and Berry (1957), exposed in elevation view, by erosion. The upper slopes show mainly extremely weathered granite (soil properties). In the lower, rock slope this material occurs between corestones and becomes progressively less abundant and is eventually absent in the outcrops close to water level.

The actual situation on this hillside is not so simple. Near the centre of the photograph, on the skyline and elsewhere at intermediate levels on the slope, hidden by trees, there are granite boulders and areas of what appear to be outcrops of essentially fresh granite.

Excavations made into slopes showing similar surface evidence have shown subsurface profiles as on Figure 2.22. At its right hand edge this figure shows a weathered profile similar to that of Ruxton and Berry (1957) except that the extremely weathered material extends locally much deeper along and next to the fault. Near the centre the profile at depth is also similar to that of Ruxton and Berry (1957). However, outcrop area A and the boulders B occur at the ground surface and are underlain by 7–10 m of

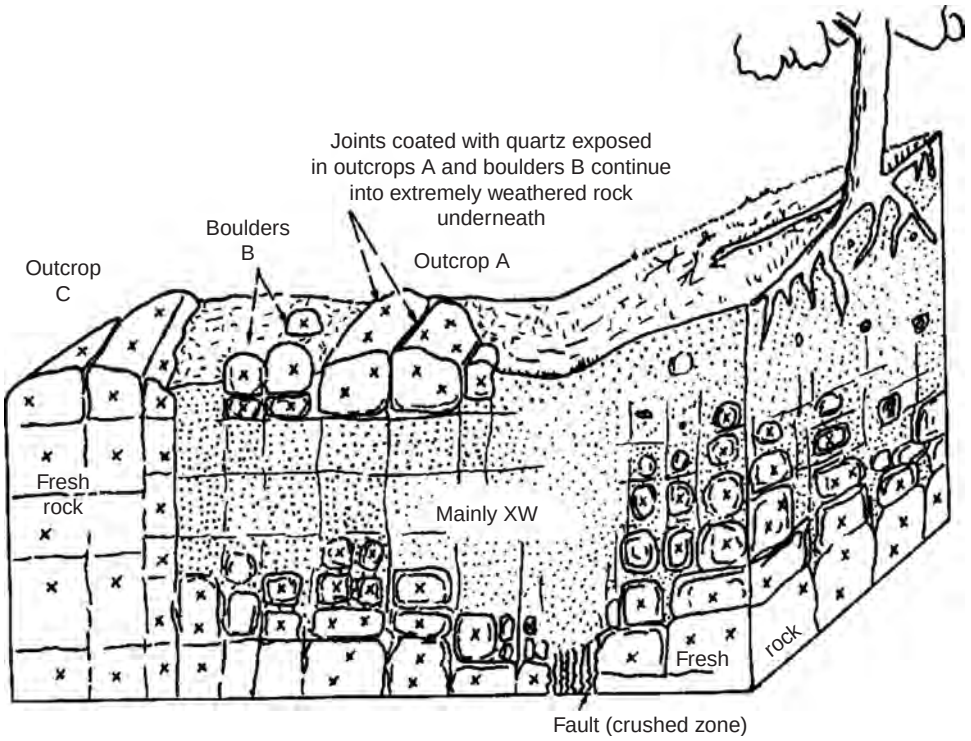


Figure 2.22 Features sometimes seen in weathered masses of granitic and other igneous rocks.

extremely weathered material. On the left side, outcrop area C looks similar to area A but is continuous downwards into mainly fresh bedrock.

The differences between the profiles A, B and C raise the following questions:

- Are outcrop A and boulders B really *in situ*?
- If they are *in situ,* then why don't they continue downwards into progressively less weathered rock mass as is the case at outcrop area C?

It can usually be shown from the continuity of the joint and fault pattern within the extremely weathered rock, whether or not masses of rock such as A and B are *in situ*. In Figure 2.22 the continuity of coated joints is clear evidence that both are *in situ*. It is often not possible to provide an unequivocal answer to the second question. One possibility is that the extremely weathered rock under A and B was partly decomposed by chemical alteration (Section 2.7) before becoming weathered. Another is that the rock substance forming A and B may be for some reason more resistant to chemical weathering than that which occurred initially adjacent to and below them. Yet another is simply that the joints which surrounded these rock masses were less permeable than those elsewhere, allowing much less groundwater percolation.

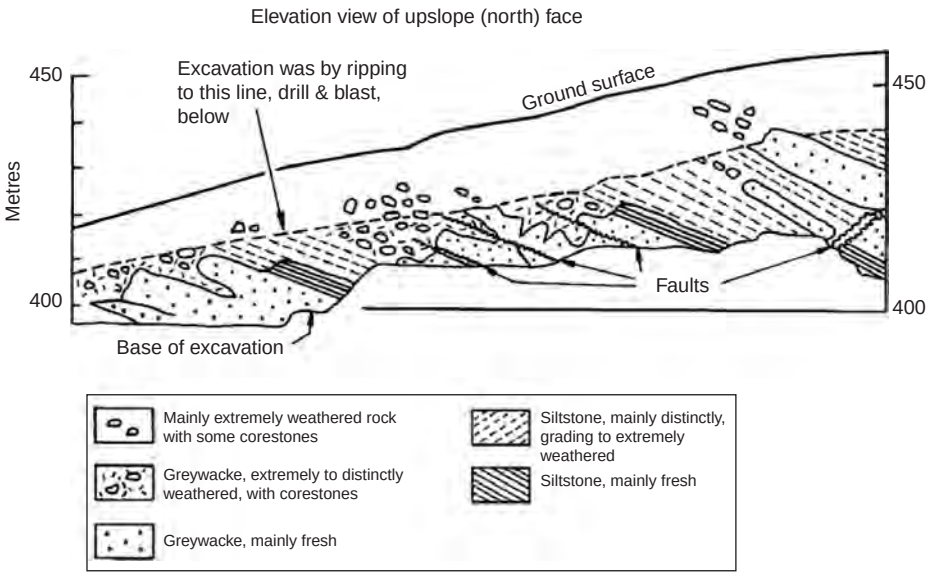


Figure 2.23 Distribution of fresh and variably weathered rock, controlled largely by variation in rock substance type, at Split Rock Dam, New South Wales.

Although it is not often possible to understand the reason for such anomalies in weathered profiles in apparently "uniform" igneous rocks, it is very important to appreciate that they occur quite commonly, and hence to expect them and allow for them in site investigations.

Figure 2.23 shows the nature of part of the weathered profile developed at the spillway of Split Rock Dam, New South Wales. The spillway excavation below the depth of ripping refusal was the designated quarry to supply rockfill for the dam. Alternating beds of greywacke, greywacke breccia and siltstone dip gently into the hill slope. The greywacke is extremely strong when fresh and relatively resistant to chemical weathering. The siltstone is strong to very strong when fresh, but is more susceptible to chemical weathering than the other rocks. The resulting profile, with beds of fresh greywacke (extremely strong) overlying variably weathered siltstone (weak rock grading to soil properties) made it difficult to meet some rockfill requirements, with normal quarrying methods.

2.6.4.3 *Time and erosion*

The depth of a weathered profile depends upon the amount of time during which the rocks have been exposed. However, this factor (time) must be considered together with erosion, because erosion will have been continually lowering the ground surface, during the development of the profile. In tectonically stable areas where the effects of erosion have been relatively small, great depths of weathered rock have developed. For example, large areas of Western Australia which have been relatively stable and mainly exposed since Precambrian time, are now almost flat "peneplains", underlain

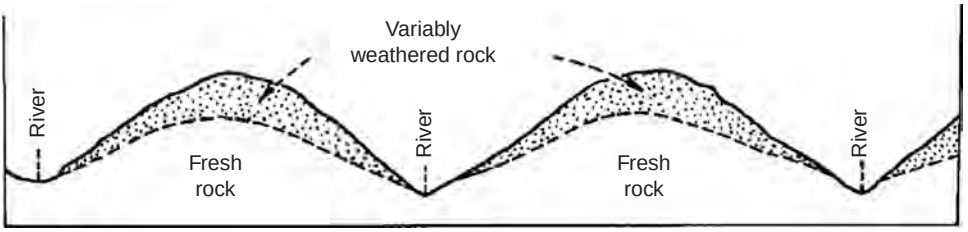


Figure 2.24 Shape of upper surface of fresh rock in a situation where the rate of valley erosion has been exceeding the rate of lowering of the chemically weathered zone.

by extremely weathered rock to depths of 30 m to 50 m. In tectonically active areas uplift and subsequent erosion occur, and result in much shallower weathered profiles in comparable rocks. In the areas of North America and Scandinavia eroded by ice-sheets during the Pleistocene glaciation, there is generally no chemically weathered profile.

Figure 2.24 shows a situation which is frequently seen: weathered profiles which are deepest beneath hilltops, ridges and plateaus, and shallow or absent at or close to the floors of valleys. This situation may have arisen simply because the rate of valley erosion (i.e. removal of weathered materials) has exceeded the rate of deepening of the weathered profile. However, in some places situations similar to that in Figure 2.24 can have a different origin as indicated in Figure 2.25. This shows the situation in parts of southern and eastern Australia, where mountain ranges were formed during late Tertiary time, by the faulting and uplifting of deeply weathered rock. Relatively rapid erosion of river valleys since late Tertiary time has resulted in composite weathered profiles as shown on Figure 2.25. The rock near the valley floors is mainly fresh but is usually somewhat mechanically loosened due to destressing.

Ancient weathered profiles can be buried and preserved beneath younger sediments or even sedimentary rocks. Figure 2.26 is a diagrammatic cross section showing the geological situation at Yellow Pinch dam in New South Wales. The valley walls at this site are formed by a near-horizontal sequence of mainly conglomerates and sandstones, of Permian age. These rocks are slightly weathered. However, at and just below river level, they unconformably overlie granodiorite of Devonian age. The granodiorite is moderately to highly weathered in its uppermost 2–5 m. The upper surface of the granodiorite is an eroded and weathered land surface of Permian Age.

2.6.4.4 *Groundwater and topography*

Optimum conditions for chemical weathering occur where vertical movement of aerated groundwater is at a maximum, i.e. beneath elevated flat ground or gentle slopes, above the main water table. Beneath steep slopes there is less infiltration of surface waters due to rapid runoff.

Field evidence in many places indicates that very little weathering has occurred beneath the main or fundamental water table. This is probably due to less oxygen and slower ground-water movement, below the water surface. In some places it is due to the fact that the main water table is in fact perched above fresh rock which is effectively impervious. However, where suitably permeable rocks occur beneath the

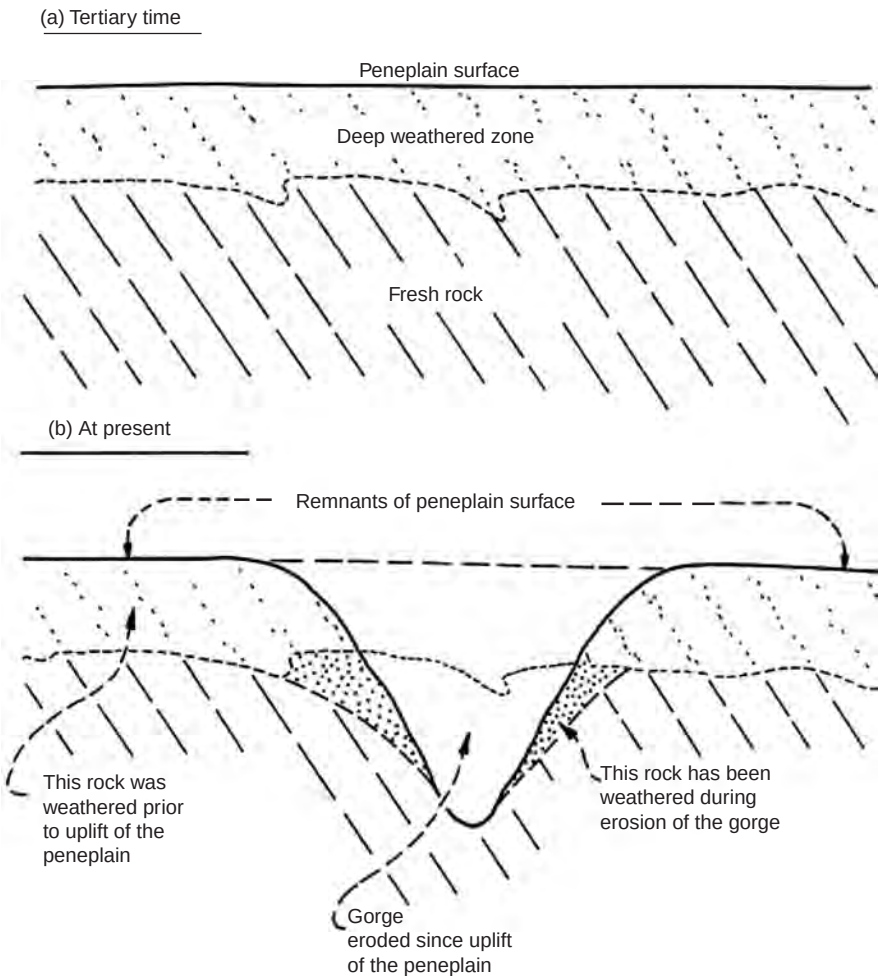

(a) Tertiary time
Peneplain surface
Deep weathered zone
Fresh rock
(b) At present
Remnants of peneplain surface
This rock was weathered prior to uplift of the peneplain
This rock has been weathered during erosion of the gorge
Gorge eroded since uplift of the peneplain

Figure 2.25 Type of valley weathered profile where a river has cut down through an uplifted block which has been deeply weathered previously.

main water table, rapid groundwater flow can occur well below the water table and chemical weathering (or solution, in soluble rocks) occurs.

It must be appreciated that due to changes in climate, the present water table may be higher or lower than the water table which dominated when a particular weathered profile was formed.

2.6.4.5 *Features of weathered profiles near valley floors*

Figure 2.27 is a hypothetical cross section through the lower part of a relatively steep valley in relatively strong, jointed rocks. It shows diagrammatically a number of features found commonly when excavating foundations for dams.

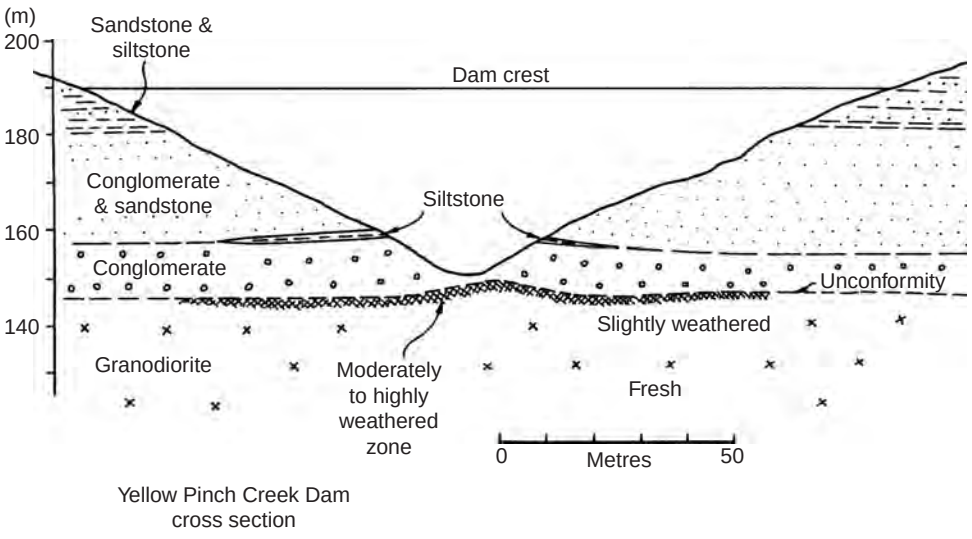


Figure 2.26 Cross section at Yellow Pinch Dam, New South Wales, showing buried land surface with weathered zone.

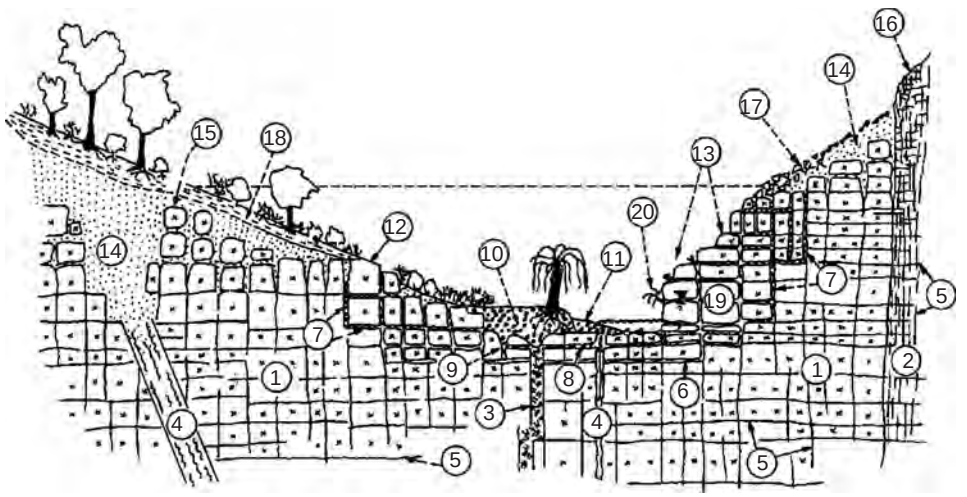


Figure 2.27 Diagrammatic cross section through the floor and lowest slopes of a steep-sided river valley, showing the types of feature developed by mechanical and chemical weathering processes.

The river is flowing in an entrenched meander channel and at this site it has been migrating towards the right bank. As a result of this the left bank slopes are flatter and show less rock outcrop, than the right bank slopes. The river channel occupies only half of the valley floor, the left side being formed by a well established (overgrown) alluvial terrace.

There are three bedrock types: granite (1) and microgranite (2) which are fresh and very strong at depth, and basalt (3) in the form of an intrusive dyke which is distinctly altered, weak. The granite contains two faults (4) in which the rock is sheared and partly crushed to soil properties. Joints at depth (5) are mainly tightly closed, but as the valley floor is approached they are either slightly open (6) due to destressing or else (7) have been open but are now partly or wholly infilled with clay which has migrated down from the chemically weathered zone. In the bedrock just below the river, some joints (8) have become infilled with alluvial silt which has migrated down from the river.

It can be seen that although the valley in profile is V-shaped, the profile of the base of the main mechanically weathered zone is closer to U-shaped. This is often found to be the case and is due to pronounced destressing at the highly stressed base of the V-notch (see Figure 2.11a). In this very strong rock there is no obvious bulge or antiform structure across the valley floor, but the opened joints beneath and next to the river bed (including that part beneath the alluvial terrace) indicate that significant rebound movements have occurred.

The actual river bed below the alluvium (10) and (11) contains a slot (9), eroded differentially by the river, along an infilled joint. Similar eroded slots occur over the altered dyke (3) and fault (4).

Upslope from the river there are small granite outcrops (12) on the left bank and larger and steeper granite outcrops (13) on the right bank.

Above the levels of these outcrops the granite is variably weathered. Extremely ranging to distinctly weathered rock (14) locally contains corestones (15) of fresh and slightly weathered rock, and extends downwards next to some joints in the dominantly fresh rock. Extremely weathered rock extends down to about river level, along the fault zone (4).

The microgranite is very resistant to chemical weathering, and forms fresh and slightly weathered steep outcrops (16) at the ground surface. This rock has closely spaced joints, however, and is mechanically weathered near the surface. Toppled and fallen blocks of microgranite have formed the scree (17) which covers the slope below.

On the left bank, slopewash soil (18) derived from extremely weathered granite occurs to shallow depth beneath the ground surface in most places, becoming deeper locally over the fault (4) and beneath gullies.

The water table (19) on the right bank, daylights at a seeping, partly infilled joint. Ferns and swampy type grasses (20) are established along the outcrop of this joint.

The alluvial gravels (10) beneath the terrace are very old (of Pleistocene age, probably). Gravels of this age are commonly partly weathered. Sands and gravels of the present day point bar (11) are not weathered – they are deposited during the dying stages of present-day floods.

2.6.5 Complications due to cementation

Chemical weathering does not always result in weakening of rock substances. Detailed observations at many sites have indicated that cements such as limonite or other iron oxide minerals have been leached out from one zone, to produce weaker, more porous substance, and redeposited in another to produce stronger, denser substance. The latter substance in some cases has been noticeably stronger than the (assumed) fresh

substance. Joints and faults have also been found to have been strengthened greatly by such deposition during weathering.

These effects are seen commonly in all rock types in the deeply weathered mantle of western and southern Australia and in many siltstones and sandstones in Victoria and New south Wales. They are believed to result from weathering during warm, humid, ranging to tropical conditions.

Similar effects have been observed in siltstones and sandstones containing small amounts of carbonate minerals in their matrices.

2.7 CHEMICAL ALTERATION

Some igneous rocks have been partly or wholly decomposed by hot waters and gases during late stages of their solidification. The hot waters may be derived from the igneous magma or else they may be groundwaters which have penetrated to great depth and become heated by the magma. Sedimentary and metamorphic rocks in the vicinity may also become partly or wholly decomposed (Figure 2.14).

This type of deep-seated decomposition is called hydrothermal alteration. It involves the chemical breakdown of some of the primary (original) minerals to form secondary minerals, which are generally weaker and less stable in water, than the primary minerals. The secondary minerals include serpentine and montmorillonite (from olivine), chlorite (from biotite) and kaolin (from felspars). The altered rocks are therefore generally weaker, and more susceptible to chemical weathering, than the same rocks when unaltered.

Altered rocks can often be distinguished from weathered rocks by their colour – usually green or white, whereas weathered rocks are usually yellow-brown or brown due to oxidation which has produced limonite in most of them. It is important in dam engineering to be able to distinguish between altered and weathered rock. This is because in an altered rock situation the altered materials may be overlain by fresh rock and become progressively more altered and weaker, with increasing depth. It is possible to find both altered and weathered rock at the same site, with some of the weathered rock being previously altered (See Section 2.6.4.2 and Figure 2.22).

2.8 CLASSIFICATION OF WEATHERED ROCK

From the discussion in Sections 2.5 to 2.7 it is not surprising that the products of rock weathering are often highly complex and variable over short distances. To assist in mapping and understanding them, some simplification and classification into groups with like characteristics has been required. Two main types of classification have been developed:

- Rock substance types, which define fresh substance and 4 or 5 "grades" of weathered substance, each grade showing greater effects of weathering than the previous one: examples include Moye (1955) and McMahon *et al.* (1975);
- Rock mass types, in which rock masses are classified into zones showing progressively greater weathering effects, as indicated by the percentages of various

grades of weathered rock substances within them: examples include Ruxton and Berry (1957), Deere and Patton (1971), International Society for Rock Mechanics (1978) and British Standards Institution (1981).

The substance type systems recommended by the authors are set out below in Section 2.8.1. The authors consider that mass type classifications are best designed specifically for individual sites, because they need to be tailored to the project needs as well as the site conditions. In practice, sites for dams are usually classified into zones taking into account variations in the overall structure (rock types, folds, faults etc) of the rock mass, and the nature and distribution of at least the following:

- mechanically loosened rock mass,
- infilled seams and
- rock substances at various weathered grades.

The Geological Society Engineering Group (1995) provides useful reviews of various aspects of rock weathering and of classification systems for weathered rocks. It highlights the difficulties which arise in classification and concludes that any attempt to devise a single standard system to cover all rock types would be "futile". It includes a flow chart containing a suggested classification system or "Approach", for each of four different weathered rock situations. The explanatory paragraphs following the chart include this statement:

> "*The recommended classifications are intended as general guidelines which can form the basis for site specific or problem specific approaches*"

The authors consider that the chart can be useful if used in this way. As pointed out above, they consider that in most cases systems for weathered rock mass should be site-specific. Unfortunately the Geological Society chart is included as Figure 19 in British Standards Institution (1999), without the above note. However, in Section 44.4.2.2 the Standard notes that "*Formal classification may often not be appropriate, and so is not mandatory*".

2.8.1 Recommended system for classification of weathered rock substance

Any system should be used in conjunction with appropriate geological and engineering descriptions and should fulfill the following requirements:

1. It should provide a "shorthand" of descriptive terms and abbreviations, to facilitate recording (during logging and mapping) of the distribution of rock substance at various recognisable stages of change due to weathering.
2. This distribution can then be used to provide an understanding of the local (site) relationships between the intensity of weathering effects and the common controlling factors discussed in Sections 2.3 to 2.7, in particular
   - the distribution of rock substance types and
   - the pattern of defects in the mass.

Table 2.3 Recommended descriptive terms for weathered condition of rock substance. Modified from McMahon *et al.* (1975).

Term	*Symbol*	*Definition*
Residual soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered	XW	Rock is weathered to such an extent that it has soil properties, i.e. it either disintegrates or can be remoulded, in water.
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron-staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

Table 2.4 Recommended descriptive terms for strength of rock substance.

Rock strength[(1)]	*Symbol*	*Point load strength index*[(2)(3)] $I_{s(50)}$	*Approximate unconfined compressive strength Qu (MPa)*[(2)(3)]
Very weak	VW	<0.2	<5
Weak	W	0.2–1	5–25
Medium strong	MS	1–2	25–50
Strong	S	2–4	50–100
Very strong	VS	4–10	100–250
Extremely strong	ES	>10	>250

(1)Based on ISRM (1985): Suggested methods for the quantitative description of discontinuities in rock masses. (2)In rocks with planar anisotropy, the strength terms and numbers refer to the strengths when tested normal to the anisotropy. (3)The numbers assume that Qu is 25 times $I_{s(50)}$. In practice the ratio ranges typically from 10 to 25, with the weaker rocks towards the lower end.

Such understanding should assist the site investigator to make soundly based correlations and predictions about the distribution of rock in various weathered conditions in other parts of the site, not directly explored.

3. It must allow unambiguous communication of the descriptions and predictions, verbally, and in logs, drawings and reports.
4. It should enable determination of any site relationships which may exist between the weathered conditions of the rock substances present and their strengths and other engineering properties. If at a particular site such relationships can be adequately proven, it may be possible for them to be used as part of a system of acceptance criteria, e.g. for rockfill or for foundation levels.

The classification system for weathered rock substance (Table 2.3), when used in conjunction with the ISRM substance strength classification (Table 2.4) has been shown to meet these requirements, for many common rock types and is recommended

Table 2.5 Description of weathered or altered rock substance: Examples.

Rock type	*Rock condition term*	*Rock strength term*	*Abbreviated form*
Granite	Fresh	Very strong	Granite FR (VS)
Granite	Extremely weathered	Soil properties (GW-GC)	Granite XW (GW-GC)
Granite	Distinctly weathered	Medium strong	Granite DW (MS)
Sandstone	Fresh	Weak	Sandstone FR (W)
Basalt	Distinctly altered	Medium strong	Basalt DA (MS)

Table 2.6 Weathered rock substance classification for granite and similar rocks. Modified from Moye (1955) and Hosking (1990).

Term	*Abbreviation*	*Description*
Fresh	FR	Rock shows no evidence of chemical weathering
Slightly weathered	SW	Rock is slightly discoloured but rings when struck by a hammer; not noticeably weaker than fresh rock
Moderately weathered	MW	Rock is discoloured, producess only a dull thud when struck by a hammer; noticeably weaker than fresh or slightly weathered rock but dry samples about 50 mm across cannot be broken across the fabric by unaided hands
Highly weathered	HW	Rock is discoloured, can be broken and crumbled by hand, but does not readily disintegrate in water
Extremely weathered	XW	Material disintegrates when gently shaken in water, i.e. has soil properties

by the authors. Using this approach, rock substance is classified as in the examples in Table 2.5. Note that altered rock is classified in the same way as weathered rock except that "weathered" is replaced by "altered" and in the abbreviation, "W" is replaced by "A".

In some granitic and similar rocks which are very strong when fresh it is commonly possible and useful to subdivide the distinctly weathered category further, on the basis of dry strength of 50 mm diamond drill cores. The terms used are based on those of Moye (1955) and Hosking (1990), defined as in Table 2.6. The rock fabric will be apparent in all weathering grades.

The distribution of rock substance at various weathering grades can be shown on borehole logs and on plans, sections or elevation views of rock exposures, either by the abbreviations or graphically by suitable colour or black and white overprinting of the rock type symbols.

2.8.2 Limitations on classification systems for weathered rock

Quite often, particularly in sedimentary rocks, it is difficult or even impossible to determine which, if any of the rock substance at a site is fresh (unweathered). This is because such rocks can vary greatly in colour, porosity and strength due to past processes involved in their formation, rather than weathering processes.

Some rocks which are weak or very weak when fresh (e.g. some shales and poorly cemented sandstones) assume soil properties (i.e. they classify as "extremely weathered") when only slightly affected by chemical weathering. Intermediate weathered conditions cannot be defined in these cases.

Weathering products of carbonate rocks and rocks cemented by gypsum or anhydrite range from cavities or soils (due to solution) to very strong rock substances (due to redeposition of material dissolved elsewhere in the mass). Such weathering products usually cannot be described adequately using the system on Table 2.3. Site-specific classifications are preferable in such cases.

2.9 RAPID WEATHERING

Rapid weathering processes are defined here as those processes which cause exposed rocks to be significantly weakened during periods of only days, months or years. In general the rocks most affected by rapid weathering are relatively porous and not of high strength. However even some very high strength rocks can be affected and so careful observational studies and testing are needed whenever rocks are to be used as construction materials.

2.9.1 Slaking of mudrocks

The weakest claystones and shales, which can barely be described as rock, have usually been strengthened only by consolidation. When such "compaction shales" are exposed in excavations they usually develop fine cracks at the exposed surface due to destressing and drying out. Swelling, further cracking and sometimes complete disintegration back to clay occurs, when the destressed shale or claystone is allowed to absorb water.

Stronger claystones, siltstones and shales (i.e. in the weak to strong rock range) have usually received some strength by cementation and/or recrystallisation of clay minerals to form micas, as well as by consolidation. These rocks also generally develop fine drying out cracks when exposed by excavation but the cracking is much less severe and occurs more slowly than in the "compaction shales". Once the fine cracks appear on the rock surface, addition of water causes relatively rapid deterioration of the rock. The reasons for this are believed to be as follows. Water is absorbed rapidly into the cracks and adjacent rock substances, by capillary suction. The water compresses the air trapped in the cracks and allows clay minerals in the adjacent rock to swell slightly. As a result the cracks widen and propagate.

The mechanisms of slaking are discussed in more detail by Taylor and Smith (1986), Taylor (1988) and Olivier (1990).

Many strong siltstones develop cracks eventually with repeated wetting and drying. However in constant humidity environments such as in rockfills or earthfill, such rocks have been found to have suffered little or no breakdown over periods of up to 90 years.

The weathering of mudrocks and its significance in dam construction is discussed in Chapter 3, Sections 3.5.1 and 3.5.4.

Some poorly cemented sandstones which contain a high proportion of clay minerals in their matrices, show similar slaking properties to mudrocks.

Laboratory tests aimed at distinguishing between durable and slaking rocks include the ISRM Slake Durability Test (ISRM, 1979) and the Modified Jar Slake Index Test (Czerewko & Cripps, 2001).

2.9.2 Crystal growth in pores

Where rock is exposed in saline environments, e.g. as rockfill or riprap on marine breakwaters and some tailings dams, some salt is absorbed by the rock either by periodic inundation or from spray. During warm, dry conditions the rock dries out and salt crystals grow in pores near its surface, disrupting mineral grains (as in the Sodium Sulphate Soundness Test). Periodic wetting up with rainwater causes further disruption. It appears that fresh water is drawn rapidly into the pores by osmotic and capillary suction, compressing entrained air. The amount and rate of degradation which occurs by the above process depends largely upon the porosity, texture and strength of the rock.

2.9.3 Expansion of secondary minerals

Some rocks appear fresh and strong but in fact have been chemically altered (see Section 2.7) and contain highly expansive secondary minerals. If these are in sufficient quantities they can cause the rocks to be significantly weakened or even to disintegrate on inundation or on exposure to the weather. The rocks most commonly affected are basic igneous rocks (basalt, dolerite and gabbro) and the most common expansive secondary mineral is montmorillonite (see Chapter 3, Section 3.2.3).

2.9.4 Oxidation of sulphide minerals

Metallic sulphide minerals (e.g. pyrite) occur in small amounts in many rock types of all ages. They can occur in many ways, the most common being:

- grains from microscopic to a few millimetres across, scattered through the rock,
- grains as above, concentrated within particular beds,
- veins of any size and
- coatings on joints or other defects.

In outcrops and in weathered rock, sulphide minerals are invariably absent, having been oxidised to form limonite or similar minerals. Due to their lack of surface exposure, small grain sizes, and often complex distribution, the sulphide mineral content in rock is usually difficult to quantify and sometimes not recognised.

When exposed to oxygen and moisture, e.g. in excavations or in rock processed for rockfill, filters or concrete aggregates, sulphides can oxidise rapidly and release sulphuric acid and metallic sulphates. Bacteria assist in oxidation. The acid can attack other minerals in the rocks, weakening or dissolving them and producing other sulphates which can cause swelling, heave and sulphate attack on rock or concrete (see also Section 3.5.1, Engineering properties of mudrocks).

The effects listed above can render sulphide-bearing rocks unsuitable for rockfill, filters or concrete aggregates. Also, acid and metallic salts can be released into the

environment. Taylor (1998) describes serious problems of this kind, referred to in the mining industry as "acid drainage". This has occurred in many places due to oxidation in waste rock dumps and tailings from mining operations. Abandoned mine excavations which exposed sulphide minerals have also caused acid drainage problems.

The severe effects described by Taylor (1998) have come mainly from materials with high contents of sulphide minerals. However the following examples indicate that oxidation of much smaller contents of sulphides in rocks used commonly in dam construction may produce undesirable effects.

2.9.4.1 *Sulphide oxidation effects in rockfill dams – some examples*

Corin Dam (Australian Capital Territory). Sandstone and quartzite which formed most of the rockfill and filters in this 75 m high earth and rockfill dam contained about 1% of pyrite (Haldane *et al.*, 1971). The reservoir started to fill in 1968, with water of pH about 6.5. During and after filling, seepage from the downstream rockfill was found to be highly acidic (pH around 4). Both acidity and conductivity were highest after heavy rainfall. It was concluded that oxidation of pyrite in the fill was occurring due to the percolation of rainfall as well as underseepage from the dam. Haldane *et al.* (1971) reported that the acid waters (base flow rate about 25 l/s) were diluted sufficiently by other waters to prevent any problems downstream.

Testing of the reservoir water in the 1980's and 1990's showed it to be slightly alkaline and to have an average sulphate content of 0.6 mg/l. Sampling of the seepage showed that there had been a gradual decrease in its acidity, pH being around 5 by March 1999 (Tabatabaei, 1999, and Tabatabaei, Pers. comm.).

Kangaroo Creek Dam (South Australia). Under Section 2.4.4 decomposition of sulphide minerals, Fell *et al.* (1992) reported on observed oxidation of metallic sulphides and associated fretting in schist exposed in excavations at the site of this concrete-faced rockfill dam. During the early site studies the schist was judged to contain less than 1 percent of sulphides. Based on the absence of deterioration in 40-year old schist rockfill near the site, the schist was adopted as the main source of rockfill, with some dolomite in the uppermost quarter of the bank (Trudinger, 1973). It was assumed that pyrite oxidation would not occur in the constant moisture environment at depth in the dam.

As at Corin Dam, the underseepage includes rainwater which enters the rockfill. The seepage rate fluctuates widely with rainfall and storage level, but is usually between 0.5 l/s and 2 l/s. Chemical analyses of the seepage have been made only since 1999 (29 years after first storing of water). The seepage has a pH of around 7 (slightly lower than that of the reservoir) but much higher concentrations of calcium and magnesium sulphates than the reservoir water. These concentrations have been decreasing since 1999 and it can be inferred that they were higher during the early years of storage. It is considered that oxidation of the pyrite and acid attack on the dolomite, have been occurring since completion of the dam in 1970. However, the embankment is performing satisfactorily, and the effects of the seepage on the quality of the water downstream have not been significant.

Carsington Dam (Derbyshire, UK). This dam failed by upstream sliding during construction, in 1984. It was a 38 m high zoned rockfill structure (Cripps *et al.*, 1993). Its clay core (extremely weathered mudstone) contained 0.8% pyrite and 0.25% calcite. The shoulders, of moderately weathered mudstone, had 5% pyrite and 7% calcite.

Drainage blankets were of limestone. The dam was reconstructed between 1989 and 1991. During the reconstruction it was revealed that sulphuric acid generated from the pyritic fills had attacked the limestone and resulted in blockage of the drains by precipitates of gypsum and iron hydroxide. Also carbon dioxide had accumulated in drains, manholes and excavations. Seepage from the dam had high concentrations of sulphates, iron and other metals. While these features had not been responsible for the failure, they were undesirable and the new design included the following features aimed at reducing or counteracting the effects of pyrite oxidation:

– Allowance for degradation, in the embankment shear strength parameters,
– Non-calcareous fill used for the drainage blankets and filters,
– Concrete structures protected by coatings of bitumen,
– Stringent safety precautions against gas in confined spaces,
– Downstream lagoons for long-term treatment of seepage waters.

Roadford Dam (Devon, UK). This is an asphaltic concrete faced rockfill embankment 45 m high. It was built in 1988–89 (Cripps *et al.*, 1993). The rockfill materials are sandstone and mudstone with average pyrite content of 1.12%. A basal drainage blanket was formed by altered dolerite, which contained 11% calcite. Some degradation of the fill was allowed for in the selection of shear strength parameters and of freeboard.

On first filling in 1989, drainage water was found to have unacceptably high levels of sulphate, calcium, iron and other minerals. Rapid pyrite oxidation and sulphuric acid attack on the calcite was indicated. From the results of tests on the rockfill and blanket materials about 30 months later, Cripps *et al.* (1993) conclude that the amounts of calcite removed were so small that the effect on the geotechnical properties of the materials would be negligible. However they note that the drainage water requires treatment, probably "for some years".

2.9.4.2 *Possible effects of sulphide oxidation in rockfill dams*

The four cases discussed above indicate that if materials used in rockfill dams contain sulphide materials, the following can occur:

1. Seepage waters containing sulphuric acid and metallic sulphates, which may require ongoing remedial treatment,
2. Attack on concrete by sulphuric acid and/or sulphates,
3. Lowering of strength of individual particles in rockfill, giving rise to settlement or lowered shear strength of the embankment.

If an embankment also contains carbonate rocks or minerals, in its rockfill, drains, filters, or in its foundation, these could be attacked by sulphuric acid to cause any of the following:

4. Clogging or cementation of drains or filters, by sulphate minerals,
5. Loss of material and changes to gradings, in carbonate rockfill and/or filter zones, giving rise to settlement and/or impaired filter performance.

Whether or not any of the above effects occur to an extent affecting the successful operation of a project would depend on the interaction of many site-specific factors. It is possible also that some of the above effects could occur when a dam of any type

is built on a foundation (or with a storage area) containing sufficiently large amounts of sulphide minerals. See further discussion and recommendations in Section 3.7.7.

2.9.4.3 *Sulphide oxidation – implications for site studies*

In the light of all of the above it is important that site investigations for dam projects include checking for the presence of sulphide minerals and assessment of the risk of possible problems associated with their breakdown. Such studies should include:

(a) checking for any known sulphide mineralization or acid drainage problems in the region,
(b) checking for sulphides and for past and current oxidation effects, during geological mapping and during logging of test excavations and drill samples,
(c) testing streams and groundwaters for pH, electrical conductivity and chemical composition,
(d) examining samples petrographically to assess the types, concentrations and modes of occurrence of any sulphide minerals found during (a) or (b),
(e) accelerated weathering tests or other tests to simulate the proposed in-service environment.

2.9.5 Rapid solution

Rocks which contain appreciable amounts of evaporite minerals (gypsum, anhydrite or halite) as cement or matrix, may be weakened rapidly if water is allowed to pass through them. James and Lupton (1978), James and Kirkpatrick (1980) and James (1992) provide predictive models for solution rates and guidelines for investigation of dam sites where these minerals are present. These are discussed briefly in Section 3.8.

2.9.6 Surface fretting due to electro-static moisture absorption

Within 3 years of the completion of Kangaroo Creek Dam, it was observed that fretting was occurring on the undersides of some large blocks of quartz-sericite schist exposed on the downstream face. Many of these blocks did not appear to contain sulphide minerals, and the fretted flakes did not have the characteristic taste of sulphate salts.

West (1978) confirmed that much of the deterioration of the exposed schist blocks was not caused by the sulphide/sulphate effects described by Trudinger (1973). He demonstrated that the likely cause was cyclic adsorption-desorption by the near-surface few millimetres of the schist, of moisture from the air, in a semi-confined or sheltered environment. The moisture changes resulted in expansion and contraction, probably of the sericite, in this near-surface layer.

2.10 LANDSLIDING AT DAM SITES

At the site of any dam located across a river it is quite common to find evidence of past landsliding on at least one side of the valley. This is not surprising because most river valleys will have developed by some combination of the following processes:

(a) erosive downcutting by the stream, which unloads the materials underlying the valley floor and slides and causes the valley sides to be steepened,

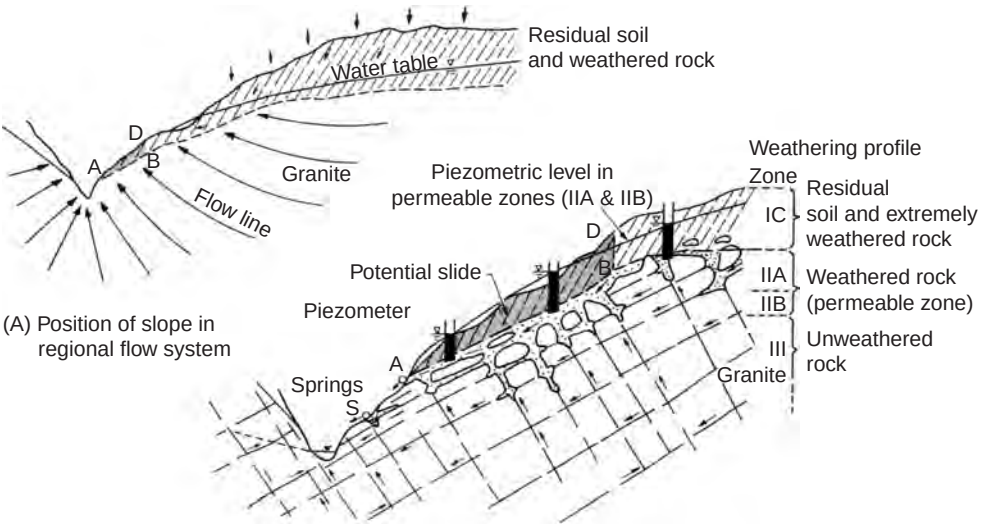


Figure 2.28 Typical conditions likely to result in landsliding of slope in weathered granite (based on Patton and Hendron, 1973).

(b) mechanical weathering processes (resulting from a) which weaken and usually increase the permeability of the materials under the valley floor and sides,
(c) chemical weathering processes which further weaken the materials forming the valley sides, converting near-surface rocks partly or wholly to soils and generally lowering their mass permeability and
(d) soil creep, erosion, deposition, and rockfalls on the valley sides, giving rise to deposits of slopewash and scree, and to overall flattening of the valley slopes.

The overall effect of (a), (b) and (c) is to reduce the stability of the valley sides.

The valley-bottom profile on Figure 2.27 was developed by these processes, in granitic rocks and in a mediterranean or semi-arid climate. Because of the relatively dry conditions and a pattern of joints which favours stability, no landsliding has occurred or appears likely to occur.

However, if the joints were in less favourable orientations and the climate wetter, resulting in a higher water table, landsliding would be more likely to occur. Figure 2.28, taken from Patton and Hendron (1973), shows such a situation.

Wedge A B D, bounded by relict joints in the low permeability zone of residual soil and extremely weathered rock, may be subject to excessive water pressures from the underlying more permeable zone, causing it to slide.

A much larger, deeper slide would be possible if a continuous thin seam of extremely weathered granite parallel to surface AB was present within the more rocky Zone IIA, day-lighting at the lower line of springs (S).

Deere and Patton (1971) describe weathering profiles developed in most common rock types and show how landsliding often occurs within them.

Many valleys (particularly in Europe and North America) have been subjected to glaciation, in relatively recent geological times (see Section 3.12). The resulting deepening and steepening has in some cases contributed to rockfills and landslides.

There are many different forms and mechanisms of landsliding and it is beyond the scope of this book to describe them. For useful accounts readers are referred to Turner and Schuster (1996), Varnes (1958, 1978), Selby (1982), Hunt (1984) and Hutchinson (1988). Stapledon (1995) provides some guidelines for geological modelling of landslides, and examples of their application.

2.10.1 First-time and "reactivated" slides

A distinction can be made between "first-time" landslide activity which develops at "intact" sites like that shown on Figure 2.28, and that which is simply reactivation or increased activity of an old landslipped mass which has become stabilised or is subject to only small occasionally active movements.

This distinction is important because there is a very large difference between the predictability and characteristics of the two kinds of sliding.

2.10.1.1 *Reactivated slides*

ICOLD (2002) has shown that in at least 75 percent of cases, disturbance of an old inactive or occasionally active landslide, e.g. by cutting into or inundating its toe, causes reactivation or increased rates of movement. Hence if there is clear evidence that all or part of a slope is an old slide mass, a high risk of reactivation by construction activities or reservoir operation must be assumed, at least until geotechnical conditions in the slope are sufficiently well known to prove otherwise.

2.10.1.2 *First-time slides*

First time slides are much more difficult to predict. Reliable prediction is not possible without considerable knowledge of the geotechnical conditions in the slope and of the nature of the possible disturbing activities. For example, there is not enough subsurface data on Figure 2.28 for one to say definitely that Wedge A B D will slide, with or without some excavation above Point A. The prediction of first time slides is discussed further in Section 2.11.

2.10.2 Importance of early recognition of evidence of past slope instability at dam sites

It is very important to recognize any evidence of past instability at a dam site, during the Prefeasibility or Feasibility and Site selection Stage of the project planning (see Chapter 4). The reasons for this are as follows:

1. Landslide debris, comprising broken rock, soil or mixtures of soil and rock in an uncompacted state is usually not acceptable anywhere in the foundation of a dam. This is because the compressibility and permeability of such materials are potentially high, variable and often unpredictable (see Sections 3.10.1.3 and 3.10.2.3). If these materials occur in only in a relatively shallow deposit, it might

be possible to remove them and expose a satisfactory foundation. However, the presence of a small deposit of landslide debris may indicate the presence of a much larger volume of unstable or potentially unstable material nearby or upslope.

2. Also as mentioned above in Section 2.10.1.1, experience shows that where landslipped materials on slopes are affected by excavation or inundation, renewed landsliding often occurs either in the materials themselves or from the source areas of the deposits. Some potential consequences of landsliding within dam storage areas are listed and discussed in Section 2.11.

In the light of the above, if all or a large part of one bank at a potential dam site is found to be underlain by landslipped materials, it is usually best to adopt, or look for, an alternative site. If it is decided to persevere with the original site, a very thorough site investigation will be needed, the results of which may well cause the site to be abandoned.

Comprehensive site studies would be necessary to determine the slide models and mechanisms and to assess the risks and hazards involved.

It should be clear that the presence of old landslides in any of situations above raises questions which ideally should be answered during the feasibility stage and certainly by the end of the design stage of the project planning (see Section 4.3).

Many old landslides are easily recognised during stereoscopic examination of aerial photographs, by their characteristic surface form and by anomalies in the local vegetation, soils or rocks (see Chapter 5).

2.10.3 Dams and landslides: Some experiences

This section describes some experiences with dams at sites showing evidence of past landsliding.

2.10.3.1 *Talbingo Dam*

At the original site for Talbingo Dam, in the Snowy Mountains of New South Wales (Figure 2.29), the right bank was formed almost entirely by the remnants of a mass of variably weathered basalt which had slipped more than a kilometre downslope and buried the valley of a tributary creek.

The geological picture in Figure 2.29 was deduced almost entirely from air photos and geological outcrop mapping on 1:2400 scale. It was confirmed by refraction seismic traverses and diamond drilling, which showed the slide mass to consist of extremely weathered basalt containing a small percentage of irregularly distributed blocks of less weathered basalt from gravel size up to several metres across.

An alternative site free of landslide evidence was adopted. At this site, more than 100 m upstream, the 161 m high Talbingo Dam (rockfill) was built, using 2.3 million m^3 of the basaltic landslide deposit for its impervious core. During operation of the borrow pit some renewed sliding movements occurred within the deposit.

2.10.3.2 *Tooma Dam*

Tooma Dam is a 68 m high earth and rockfill structure, constructed during 1958–1961 in the Snowy Mountains of New South Wales.

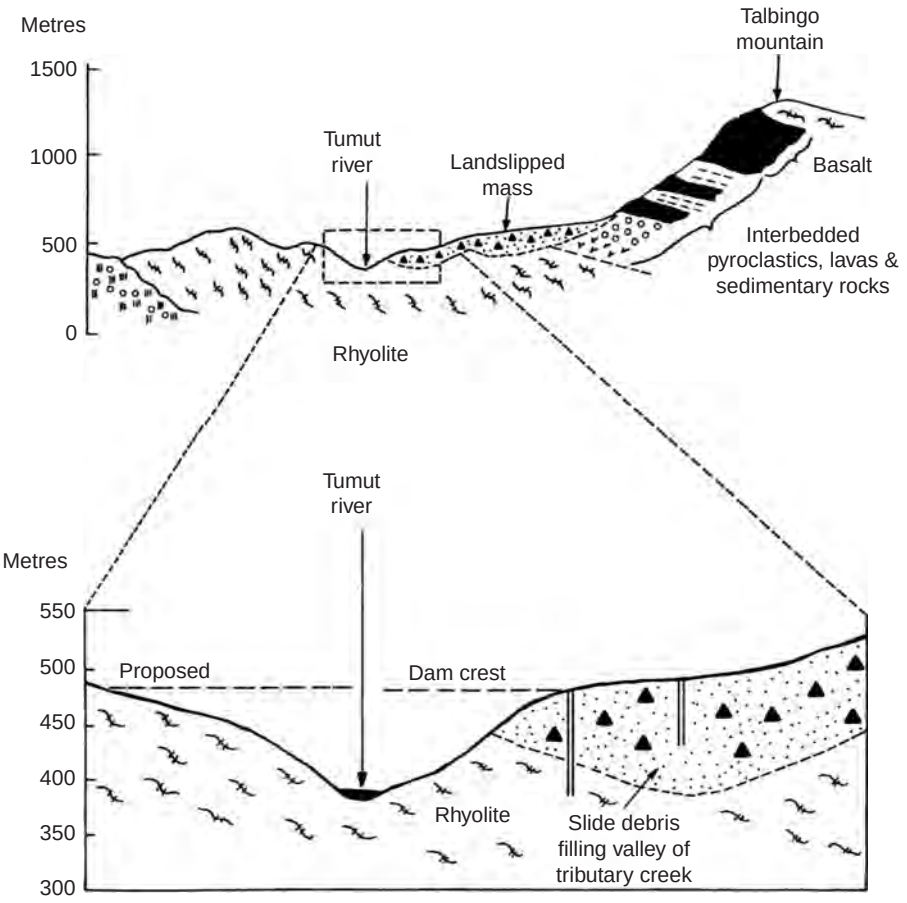


Figure 2.29 Original site for Talbingo Dam, cross section looking downstream.

The dam is located in a steep-sided valley which has been entrenched about 80 m below an older, broad valley. Prior to construction, outcrops of granitic rocks were present along both banks of the river and extended locally to 10 m to 40 m above river level. The largest outcrop area formed a cliff about 30 m high on the right bank beneath the upstream shoulder of the proposed dam (Figure 2.30 and Figure 2.31).

This outcrop and a lower outcrop area directly opposite on the left bank, were terminated on their upstream sides by soil-filled gullies. The rock outcrops above river level showed near-vertical joints roughly parallel and normal to river direction.

During the planning stages subsurface exploration of the site included trenches up to 1.5 m deep cut by hand methods, and 17 diamond boreholes with water pressure testing. It was concluded from the results that the site would show wide variations in weathered profile, but no special problems were envisaged.

During stripping of the foundation, the variations in weathered profile were confirmed (Figure 2.31). However the exposed rock was much more mechanically

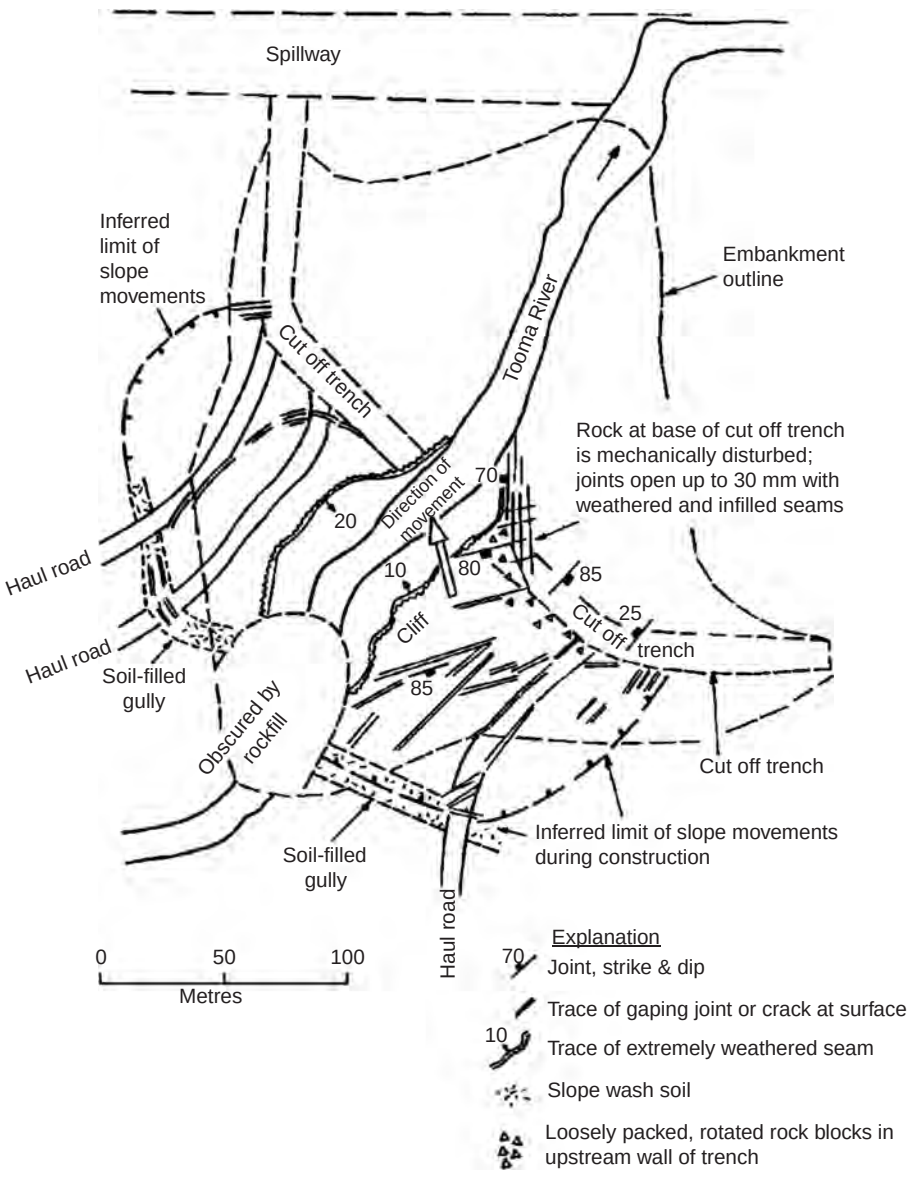


Figure 2.30 Tooma Dam, plan showing observed features and movements which occurred during construction.

loosened than predicted. In the valley sides, the floor of the cutoff trench showed near-vertical joints parallel to the river which were open as much as 30 mm, apparently due to downslope block movements (in the past) along prominent joints dipping towards the river at 10–30°. Clay-infilled joints and weathered seams next to joints were abundant.

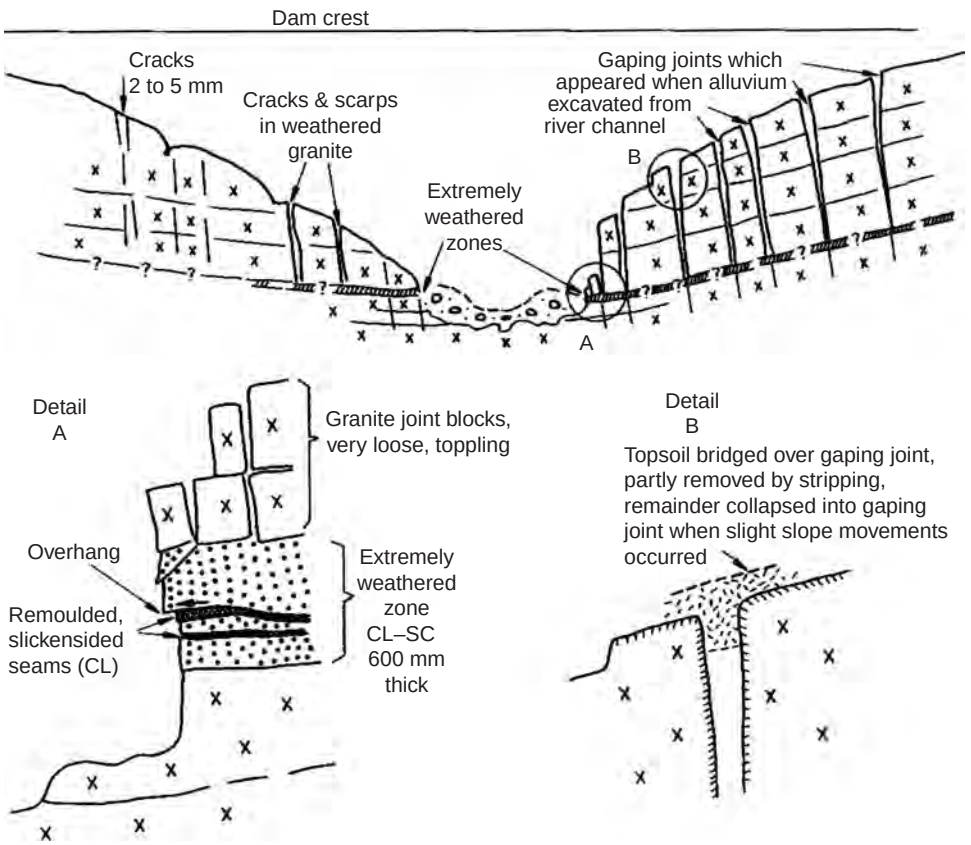


Figure 2.31 Tooma Dam, cross section and sketches through downstream shoulder.

The last specified stripping was removal of 2 m to 3 m depth of boulders and gravel from the river channel. During this operation, many cracks open from 10 mm up to 500 mm appeared in and above the cliffs on the right bank (Figure 2.30 and Figure 2.31). The cracks were mapped to the upstream side of the cutoff trench where they became ill-defined in a steeply dipping zone of open jointed, clearly disturbed and weathered rock striking obliquely to the river (Figure 2.30). Lesser cracks and scarps appeared in extremely weathered granite on the left bank.

At the same time, 400–700 mm wide zones of extremely to highly weathered granite which were wet and showed local seepages became exposed close to river bed level on both banks. Both zones were undulating but dipping generally between 10 degrees and 20 degrees towards the river. Each zone contained one or more remoulded seams which included surfaces with slickensides (Figure 2.31A). In the right bank seams these were trending downslope and obliquely downstream. Displacements of 50 mm were observed across one slickenside seam during a period of two days. Monitoring across this seam ceased when several cubic metres of rock toppled from immediately above it.

When all of these features appeared the contractor withdrew from the upstream part of the site until it could be demonstrated that large-scale landsliding was not imminent.

Close examination of the largest cracks on the right bank showed that these had formed by the collapse of surface soils into pre-existing gaping joints which appeared to extend almost down to the extremely weathered zone (Figure 2.31B). Thus, although the cumulative past downslope extension appeared to be between 1 m and 2 m, the amount of current movement appeared to be much smaller, probably just enough to cause the bridging soils to crack and fall into the already gaping joints. This was confirmed by monitoring which was started after the appearance of the cracks and showed only about 150 mm of further movement over a period of about 2 weeks.

It was concluded that the gently dipping joints were probably sheet joints and that the basal weathered zones had been formed by weathering along the lowest of these joints. The pre-construction block movements towards the river were probably initiated by stress relief and continued due to gravity.

On each bank the largest of the old movements appeared to have occurred between the upstream soil-filled gullies and the cutoff trench (Figure 2.30).

Movements of the right bank cliff area were stopped by early placement of rockfill in the river bed. Some of this is visible at the right hand edge of Figure 2.32.

Changes to the dam design were made during construction, to allow for the potential for water to penetrate along open joints under the upstream part of the earth core, to minimise the possibility of adverse deformation of the core and to allow the unstable slope to be quickly restrained. The adopted measures included:

- Steepening of the upstream face of the earth core from 1:1 to 0.25H:1V (Figure 2.33).
- Addition of a secondary downstream grout curtain, and blanket grouting between the two curtains (Figure 2.33),
- Extending the lower half of the cutoff trench about 25 m downstream, and immediate commencement of placing the upstream rockfill, in advance of the earth core zone placement.

In addition to the above, extensive dental treatment of open joints (with mortar) was carried out in the cutoff trench, and backfill concrete was placed over the badly disturbed and weathered rock at the upstream edge of the cutoff trench on the right bank, shown on Figures 2.31 and 2.33.

Further details of the above are provided in Hunter (1980) and Hunter and Hartwig (1962).

Pinkerton and McConnell (1964) describe the performance of the embankment during construction and its first 2 years of operation and show that it behaved well.

Although it was possible to modify the design during construction and a good result was achieved, there must have been some cost to the project because the evidence of past slope movements was not recognised and allowed for during the early planning. It is considered that adequate evidence was there, but was not found or recognised, mainly because the question of possible valley-side movements was not addressed. In later projects deep trenches excavated by bulldozer and ripper have usually been successful in locating the gaping or infilled joints indicative of this kind of slope instability.



Figure 2.32 Right bank of Tooma Dam after completion of foundation stripping and placement of dental concrete.

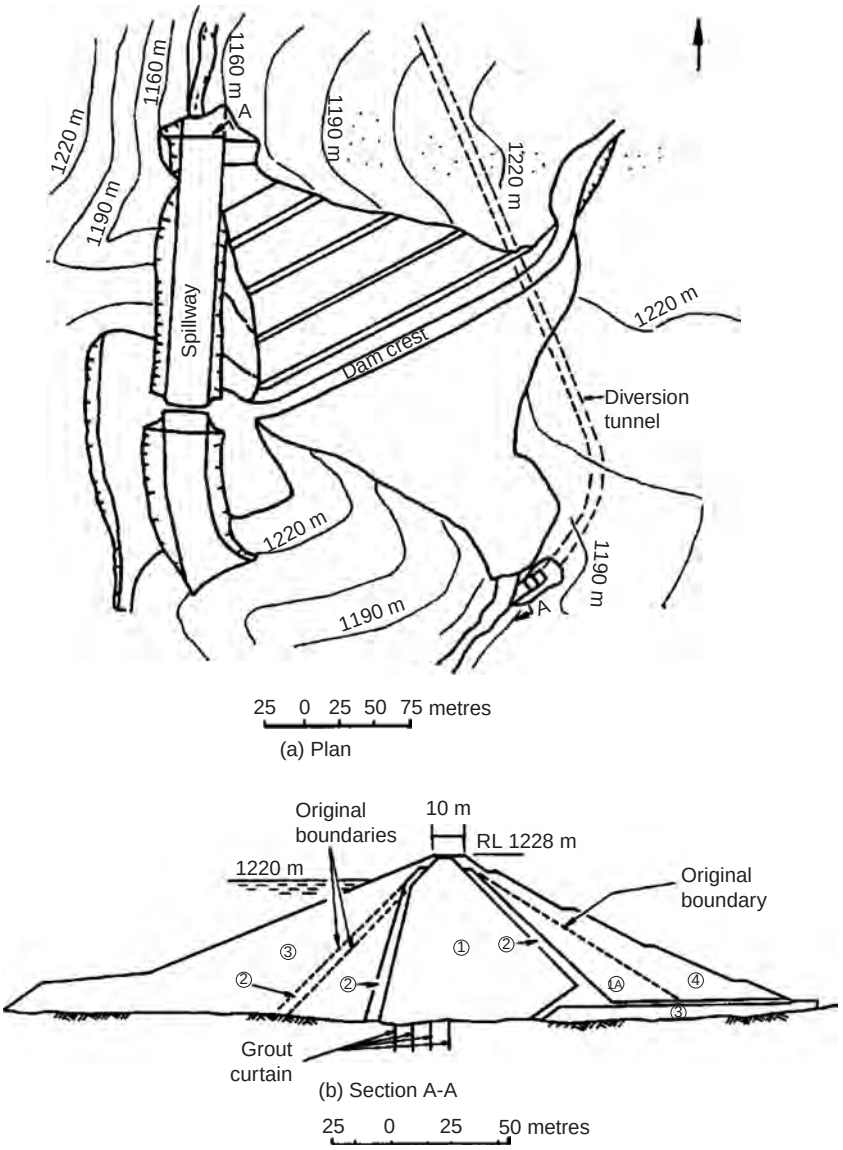


Figure 2.33 Tooma Dam, cross sections of original and rezoned embankment (Hunter, 1982).

2.10.3.3 *Wungong Dam*

Wungong Dam is a 66 m high earth and rockfill structure built during 1976–1979 near Perth, Western Australia. The site is underlain by granite containing several intrusive dykes of dolerite. The rocks are variably weathered. Lilly (1986) describes landslides which occurred during construction.

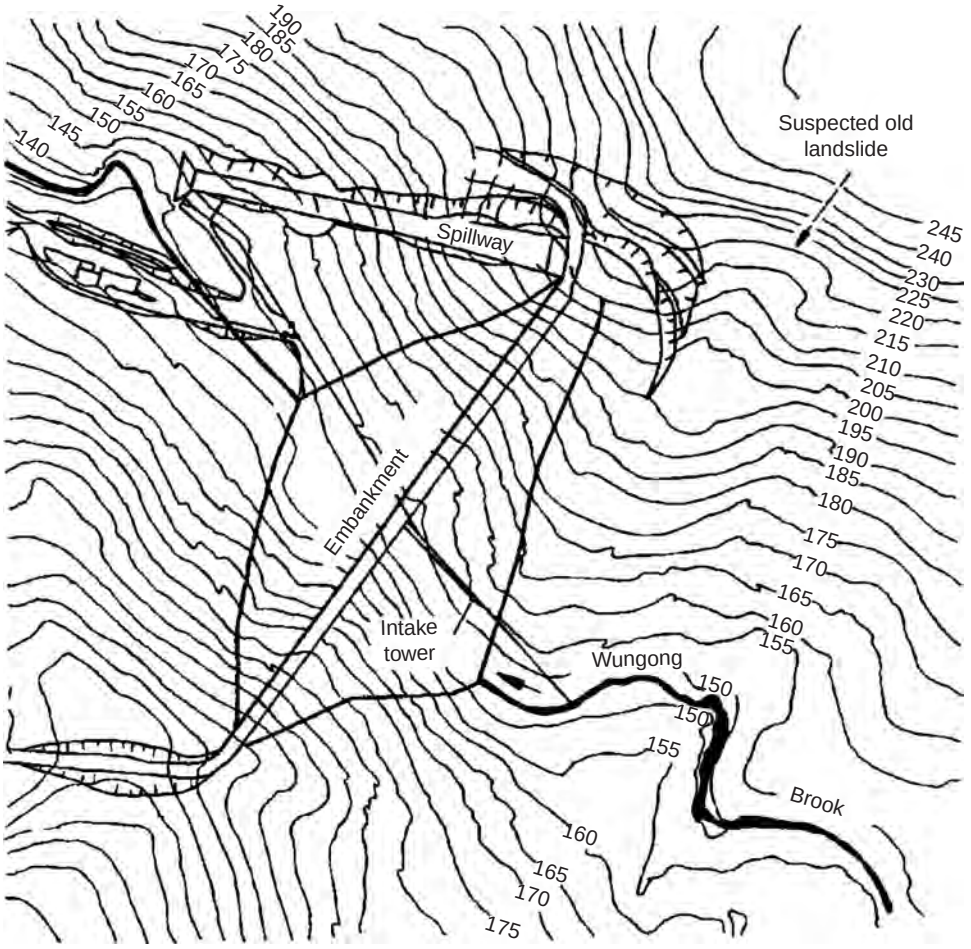


Figure 2.34 Wungong Dam, plan showing adopted layout and inferred approximate position of old landslide (Lilly, 1986).

During early site investigations, a rounded scarp was recognised on the right bank above the upstream shoulder of the proposed dam, and noted as a "suspected old landslide" (Figure 2.34). Trenches below this feature confirmed that the ground was disturbed, but the areal extent and depth of the disturbed mass was not determined.

To allow for this suspected landslide the dam axis was rotated to bring the upper right shoulder downstream of it (Figure 2.34). It was planned to investigate the area further during construction and it was expected that much of the landslipped material near the embankment would be removed during excavation of the spillway approach channel.

During stripping of the foundation and placement of fill, four landslides occurred from the area below the suspected old slide. The slides cut through a construction access

road, and the fourth and largest slide (about 100,000 m^3) cut through the spillway approach channel and moved into the upstream rockfill and filter zones (Lilly, 1986).

At this stage it was appreciated that the stability of the dam embankment was also in question and a major investigation involving 17 diamond drill holes was carried out.

Excavation of the slide material in the foundation area showed that the basal failure surface here was a seam less than 50 mm thick of micaceous clayey silt, probably a weathered and sheared, basic dyke. The seam was slightly undulating and dipping almost directly downstream (i.e. beneath the dam) at about 15°. The slide mass here was a wedge bounded by this seam and a joint striking roughly at right angles to the river, dipping 80° upstream (Figure 2.35). Movement had been directly towards the river at the wedge intersection plunge angle of about 5° (Figure 2.36, Section BB). Although water pressures obviously contributed to the sliding, an extremely low shear strength was indicated for the seam.

After removal of some rockfill which had already been placed, a 40 m by 70 m area of the rockfill foundation was excavated down to below the seam level (Figure 2.35 and Figure 2.36). The slide was then stabilised by a spur of rockfill which later formed a projecting part of the dam shoulder. Other modifications to the dam included a local widening of the downstream filter zones and placement of a filter blanket over an area of slide material left in place beneath the rockfill (Figure 2.35 and Figure 2.36).

Monitoring during filling showed local downslope movements of up to 90 mm in and upstream from the spillway approach area. Lilly (1986) explains that these were to be expected, as cracks up to 150 mm previously noted in these areas would be closing up. No significant unusual movements of the dam embankment were recorded.

Although a satisfactory dam was the end result, the landsliding and discovery of disturbed materials extending into the rockfill and filter foundations caused serious questions to be raised, and delays to construction. With hindsight, more thorough investigation and delineation of the slide during the planning stages might have resulted in the dam being located well away from the affected area.

2.10.3.4 *Sugarloaf Dam*

Sugarloaf Dam is an 85 m high, 1000 m long concrete faced rockfill structure, constructed near Melbourne, Australia during 1976–1979. It impounds an off stream storage of 95,000 megalitres.

The site geology and its influence on the design and construction of the dam are discussed in Stapledon and Casinader (1977), Casinader and Stapledon (1979) and Casinader (1982).

The site is in a region of broadly folded siltstones and sandstones of Devonian age which are distinctly weathered to depths of more than 30 m below ridges and plateaus.

Figure 2.37 shows the layout of the embankment which extends across the valley of Sugarloaf Creek and then for 500 m along the crest of a ridge. No other site was acceptable topographically.

During the feasibility stage, detailed geological mapping of surface features and trenches dug by bulldozer and ripper were the main investigation activities.

After initial excavation the floors of the trenches were cleaned carefully by small back-hoe and hand tools to ensure that all minor defects such as open and clay-filled joints and thin crushed seams were clearly exposed.

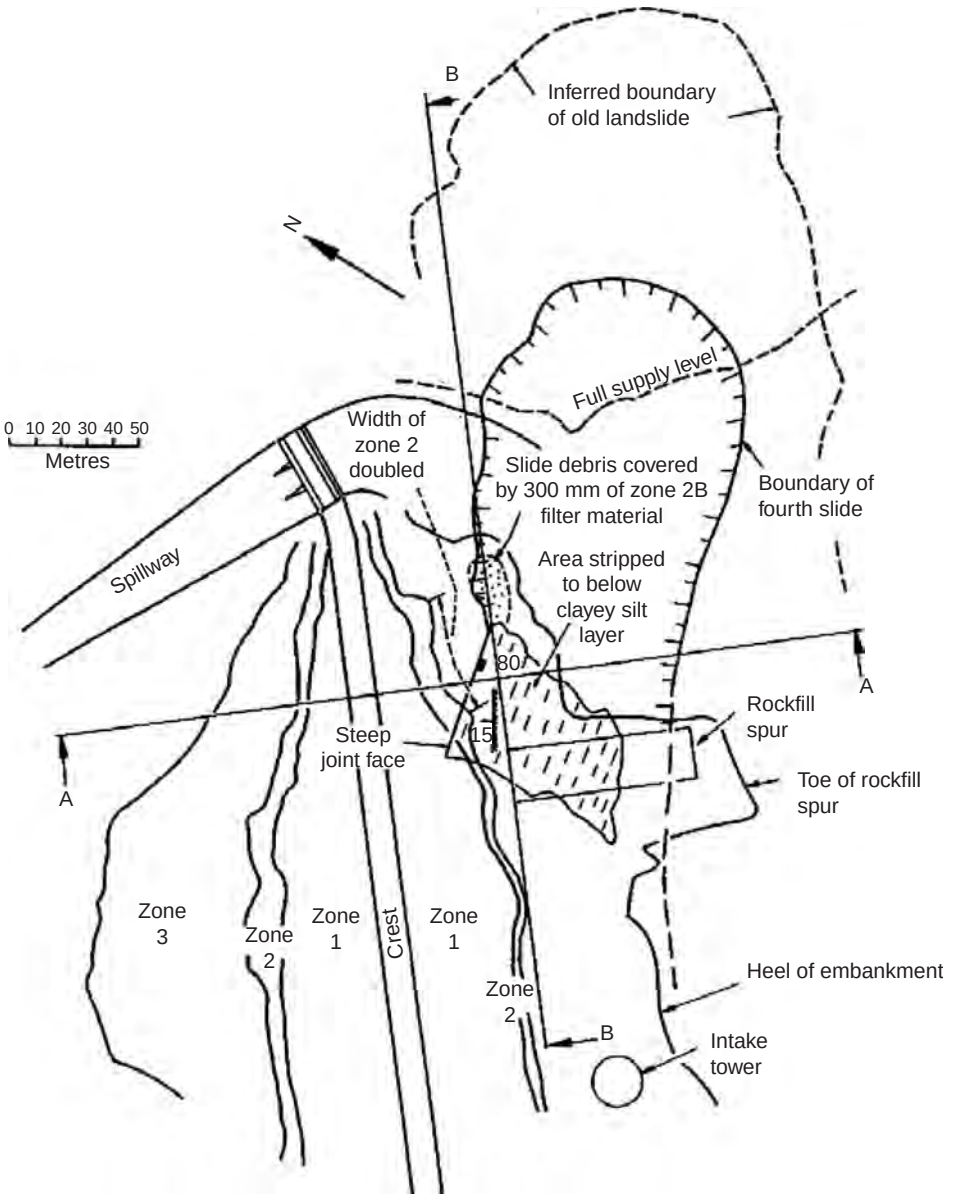


Figure 2.35 Wungong Dam, right abutment, plan showing the extent of the fourth landslide and remedial works (based on Lilly, 1986).

This work showed that much of the foundation area on both banks was formed by dipslopes; thinly interbedded and weathered siltstone and sandstone were dipping parallel to the ground surface (Figure 2.37). Dip and slope angles ranged from 12° to 30°. On the right bank a suspected old landslide was confirmed by the trenching

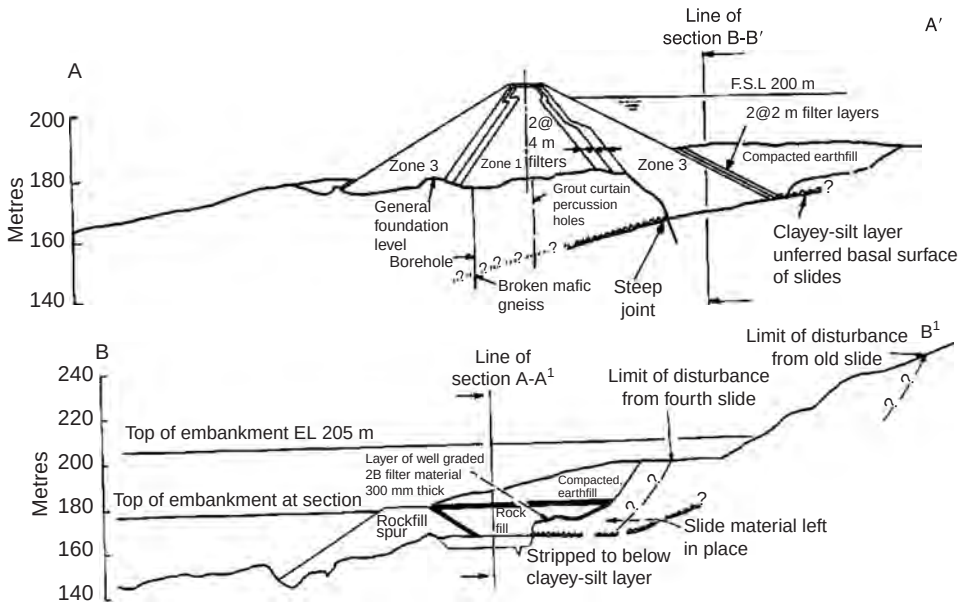


Figure 2.36 Wungong Dam, sections showing remedial works (based on Lilly, 1986).

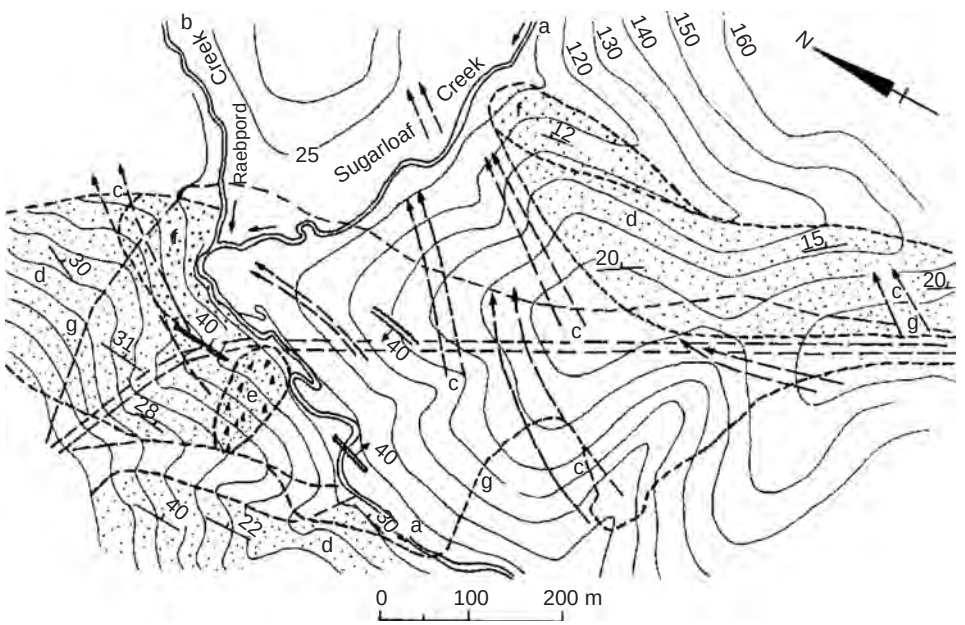


Figure 2.37 Sugarloaf Dam, geological plan showing dipslopes. (c) axes of minor plunging folds; (d) dip slope, slightly disrupted; (e) scar of old landslide; (f) dip slope, very disrupted; (g) outline of embankment.

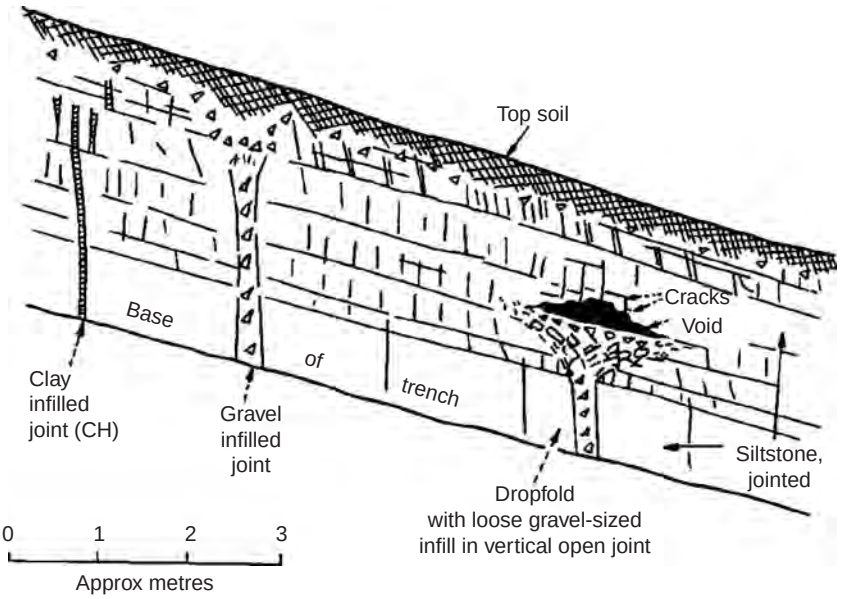


Figure 2.38 Features exposed in trenches on a left bank dip slope at Sugarloaf Dam.

to be underlain by deep, disturbed soil. With the exception of this feature the valley slopes in the foundation area showed no topographic evidence of past landsliding or of significant undercutting of the dip slopes.

Trenches cut straight down the dip slopes showed the bedding surfaces to be very smooth and that slickensided seams of low plasticity clay 2 mm to 20 mm thick were present along them at spacing of 1 m to 2 m. The seams were highly dispersive. Diamond core drilling showed that the clay seams were present to depths of more than 1 m and that they were the weathered equivalents of bedding surface faults (crushed seams, see Chapter 3, Figure 3.19 and Section 3.5.2), which were present at about the same spacing, at greater depths.

Present in some parts of the trenches were near-vertical features of the kind shown on Figure 2.38. These were mainly joints which were open or infilled with gravel or up to 20 mm of high plasticity clay. Also present near the base of a nearby dip slope was a large but ancient "dropfold" structure where upper beds had clearly collapsed into a gaping slot in a lower bed. Figure 5.12 in Chapter 5 shows this feature.

It is judged that these near-vertical and "dropfold" features had been formed by extension of the near-surface beds during downslope sliding movements along weathered bedding surface seams. Cross-cutting defects near the base of the slope and in some places past undercutting by erosion had apparently provided freedom for the slope movements to occur.

Laboratory direct shear tests showed the weathered bedding surface seams to have effective residual strengths of about 10°. The residual value was adopted in stability analyses because the seams were initially near-planar faults and the small displacements during slope movements would have been enough to reduce their strength to this

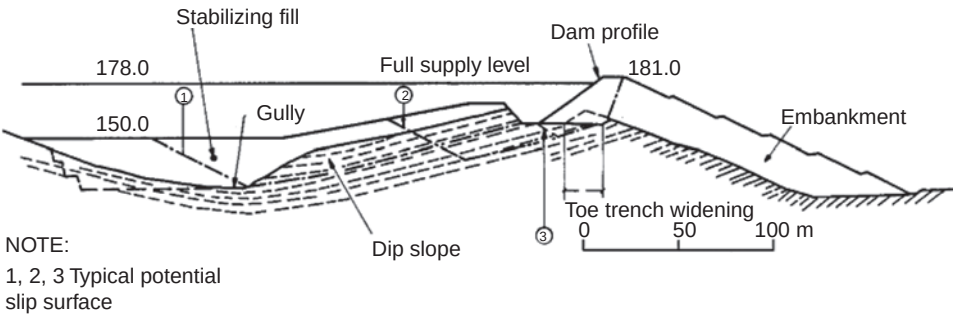


Figure 2.39 Sugarloaf Dam, cross section through left abutment ridge section.

value. The combination of very low strength dispersive seams and dip slopes provided interesting design questions which are discussed in Casinader and Stapledon (1979).

Figure 2.39 shows the arrangement adopted for the section of the dam on the left bank ridge. The plinth was located close to the crest of the ridge, to minimise loading of the dip slope. This resulted in much of the embankment being located on the downstream side of the ridge. The dip slope was stabilised by covering it and filling the gully upstream with waste material from a nearby excavation.

During construction the conditions found were essentially as predicted. Minor landsliding occurred on the right bank dip slope when the surface soil and vegetation was stripped and again when a 4 m deep trench was cut directly down the slope (Regan 1980, Stapledon 1995). These occurrences under very low normal stress conditions confirmed the need to use residual strength in the design.

Stability problems on the right bank proved more difficult than predicted, but early recognition of this during construction enabled the design here to be modified with minor disruption of the works.

The evidence of thin seams and past slope movements at Sugarloaf was quite subtle and may not have been found if it had not been consciously looked for and exposed in well-prepared, very long, deep trenches.

2.10.3.5 *Thomson Dam*

The 166 m high Thomson Dam is an earth and rockfill structure located 120 km east of Melbourne, Australia. It was constructed between 1977 and 1984.

The site is underlain by a folded, interbedded sequence of siltstone and sandstone, similar to that at Sugarloaf. However the folds are tighter and more complicated than at Sugarloaf, there are many normal and thrust faults, and bedding-surface sheared and crushed seams are spaced generally at less than 1 m intervals.

Figure 2.40 shows the major folds and faults at the site. Fold axes trend generally north-south and plunge northwards. A major feature at the site is the Thomson Syncline which plunges to the north at about 12°.

Although this fold pattern was recognised during the feasibility and pre-construction design stages, its potential to contribute to instability of slopes and foundations was not appreciated at that time.

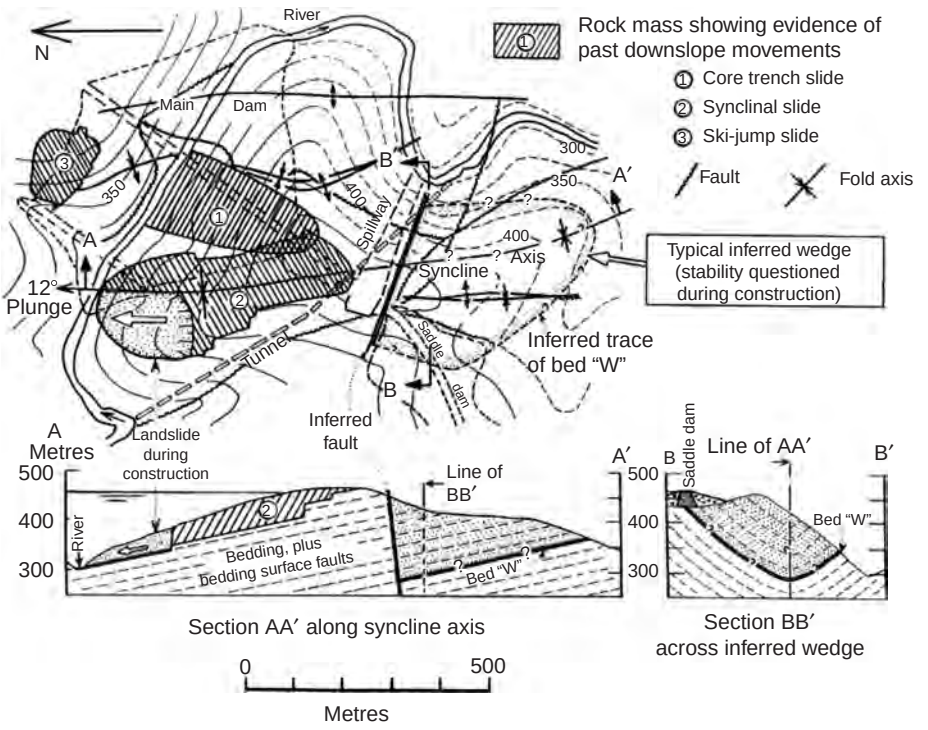


Figure 2.40 Thomson Dam site, geological plan and sections showing the main structural features.

As at Sugarloaf, the site was trenched extensively during the feasibility investigations but the trenches were not so deep and the same standard of cleanup was not achieved. However the Ski-jump slide (Figure 2.40) on the upstream left abutment was identified from highly disturbed rock in a trench cut into an area showing typical landslide topography.

Diamond drilling in the upper right bank core foundation area indicated closely jointed, seamy and partly weathered rock down to 70 m. High losses occurred during water testing of this zone and it was suspected that it may have been disrupted by past landsliding. This was allowed for in the designed depth (up to 30 m) of the cutoff trench on the upper right bank. Also, the owner, Melbourne and Metropolitan Board of Works, decided to excavate the whole of the cutoff trench on this bank, prior to awarding a contract for construction of the dam. During this excavation and excavation of associated haul roads, it was found that past landsliding in the site area had been more widespread than expected and had caused disruption of the rock mass in the three areas shown on Figure 2.40. In each area the sliding had occurred where the combined effects of valley erosion and the fold shapes had caused downslope dipping beds to daylight. Details of each slide and the nature of the disturbed materials are given in Marshall (1985).

The Ski-jump Slide proved to be a relatively small feature. The folded rock here day-lighted into a gully immediately upstream (Figure 2.40). The slide mass was

buttressed effectively by placing rockfill in the gully and the rockfill shoulder of the dam was built over it.

The Core Trench Slide proved deeper than expected, downslope movements having occurred along folded beds daylighting at river level. The trench excavation ranged from 10 m to more than 40 m deep and involved more than a million cubic metres of excavation, all by Caterpillar D9 bulldozer without ripping. In the upper half of the trench deeper excavation appeared likely to give rise to instability of the sides and so it was decided to leave the deeper, moderately disturbed rock in place and treat it with a 5-row grout curtain. This rock contained many joints which were open or clay-filled, from 1 mm to 20 mm. It was appreciated that most of the clay would remain after the grouting but could be washed out in the long term, when the reservoir was filled. To allow for this possibility a reinforced concrete gallery was built in a slot cut into the upslope half of the cutoff trench. The gallery provides access for monitoring and for drilling and grouting equipment if re-grouting should ever be needed.

The Synclinal Slide (Figure 2.40, Section AA) was discovered during construction when cracks and displacements appeared in haul roads near the toe of the ridge at the right bank upstream shoulder of the dam in mid-1977. The estimated slide mass was about 350,000 m^3, but, once the slide had been defined geologically, the potential slide mass was estimated at well over 3 million m^3, with a basal surface that would have sliced through the right abutment of the main dam and extended downstream to affect the top of the spillway. Movements in the toe area ranged from 1 mm/day in dry weather to 3 mm/day after rain. Movements stopped after 9 months (a total of 300 mm cumulative movement) on completion of a 50 m high stabilising rockfill built against the ridge toe. This required diversion of the river into a channel cut into the left bank, as the diversion tunnel was still under construction.

Detailed investigations showed that past movements in the ridge had extended right up to its crest. The sliding had occurred directly down the 12° plunging axis of the Thomson Syncline. Upslope from the toe area the base of the disturbed rock zone was stepped as shown diagrammatically on Figure 2.40, Section AA. In the ridge crest area rock disturbed by this slide occurred in the top few metres of the gate shaft (alongside and just to the left of the spillway) and also at the top of the cutoff trench. The inferred basal surface of the potential slide was estimated to be nearly 75 m below the top of the gate shaft and generally 60–70 m deep below the crest of the dam on the right abutment.

Changes to the project resulting from this slide included construction of a permanent stabilising fill of more than 2.6 million m^3, that involved filling in the valley upstream of the dam for a depth of 50 m with rockfill, constructing a larger supporting fill on the right abutment upstream from the dam and elimination of an access bridge from the gate shaft area to the upper outlet tower.

During detailed studies of the Core Trench Slide and Synclinal Slide the succession of rock types and the shape of the Thomson Syncline became well known in those areas. Projection of this known geological picture southwards raised serious questions about the possibility of first-time landsliding within the ridge which formed the reservoir rim, immediately south of the right abutment of the dam. The spillway and part of the saddle dam were located on this ridge (Figure 2.40, Section BB). Assuming a full storage and using residual strengths obtained from back analysis of the Synclinal Slide, this ridge appeared to have safety factors of less than unity against sliding downstream

along any of the numerous bedding surface seams which daylighted above river level on the downstream side. A major new site investigation was carried out over a period of 16 months, including 1.6 km of trenches, adits 250 m and 460 m long, diamond drilling and field and laboratory testing. This work is described in detail by Marshall (1985) and summarised by Stapledon (1995). It showed that irregularities in the fold shape and rotational displacements on faults effectively precluded failure of the ridge. The following additional works and changes were undertaken, to further guarantee its stability:

- Weathered, potentially unstable rock was removed from the top of the ridge,
- The axis and grout curtain of the saddle dam were moved 10 m upstream, to reduce the embankment load on the active wedge,
- An additional drainage gallery 70 m below dam crest level was driven into the ridge and connected by a drainage curtain of vertical boreholes to the previously constructed exploratory gallery that was some 115 m below dam crest level.

The site for Thomson Dam is geologically very complex and was difficult to explore due to its steepness, dense vegetation and often deep soil cover. Some evidence of the slope stability problems at the site was found during the early planning and used in the design of the dam. The owner's decision to excavate the right bank cutoff trench prior to awarding the main construction contract (Hunter, 1982) was a good one. As Hunter (1982) points out, this decision, although driven as much by concern over limiting potential industrial problems as by geotechnical aspects, was more or less in accord with the approach of Terzaghi and Leps (1958) at Vermillion Dam. Terzaghi and Leps concluded that the foundations at Vermilion were so complex that further more detailed exploration would still leave a wide margin of interpretation. They therefore advocated proceeding with construction of the dam as designed on the best interpretations and assumptions, with careful monitoring of the construction, and modification of the design to suit the conditions as found.

However, at Thomson, after construction was well advanced the expenditure of money, technical manpower and time on geotechnical site investigations far exceeded that spent in the feasibility and design stages. Most of this effort went into investigation of the downstream ridge, the stability of which was a question that should have been answered in the feasibility stage.

If the construction stage studies had found this ridge to be unsafe, the options for remedial works were very limited, extremely expensive and would themselves have required further detailed site investigations to prove their feasibility. As discussed in Section 2.11.1 and Section 4.6, the authors consider that for all dams, large and small it is vital that sufficient funds are made available at the feasibility stage to ensure that all questions affecting feasibility are asked and answered satisfactorily. For dams of the size of Thomson the exploration will often include adits as well as extensive and carefully cleaned trenches.

Dam owners, dam designers and all others involved in the implementation of a large dam project ignore the geological/geotechnical matters of their dam site and dam at their peril. All, including the geological/geotechnical team members, must ask the awkward question about potential deep-seated instability early in the process. One can only imagine the disaster at Thomson if the lingering doubts of the dam designer in

1977 about the "disturbed rock and potential instability at the dam site" had been lost in the desire to get the "job moving" and, for whatever reason or luck the 350,000 m^3 of the right abutment that did move, did not do so. That the project was set back by 5 years or so was a major concern to the owner and the designer, but that concern could be nothing compared to a major failure during initial reservoir filling, which is very likely what would have happened had the major stabilizing fill works not been constructed.

2.11 STABILITY OF SLOPES AROUND STORAGES

The slopes around a proposed storage area may include some which are intact and stable, others intact but less stable, others formed by dormant landslides and still others formed by active landslides. The local raising of the water table which occurs when the storage is filled and lowering when the storage level is drawn down may reduce the stability of any of these slopes. Sometimes the reduced stability causes landsliding in a previously intact slope. This landslide is called a "first-time" slide (see Section 2.10.1). For a dormant landslide or an active or occasionally active slide, the reduced stability can cause reactivation of the former or increased movement rates in the latter. Affected slopes are normally within the storage area (Figures 2.41 and 2.42). However, slope failure is also possible (but rare) on the outer side of a storage rim (Figure 2.43).

The consequences of landsliding within the storage area can include

- Damage to existing property or infrastructure,
- Constraints on operation of the storage,
- Damage to roads,

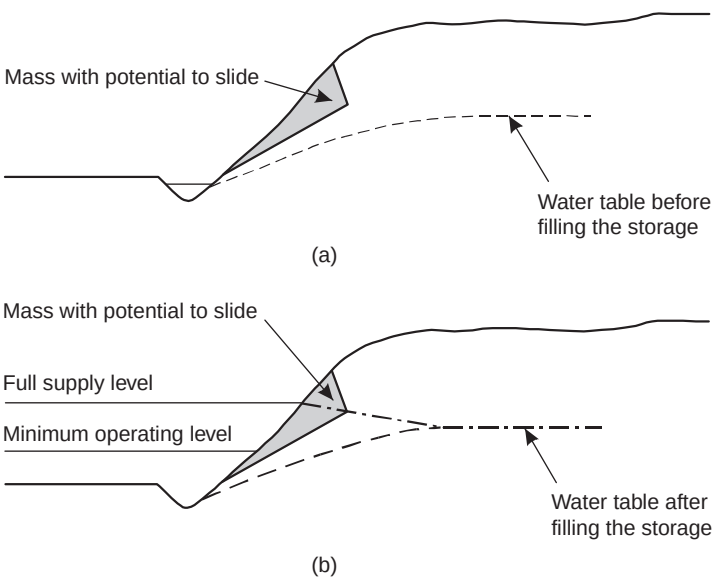


Figure 2.41 Effect of filling and operating a storage, on the stability of a jointed rock slope partly inundated by it. (a) Before filling the storage; (b) During operation.

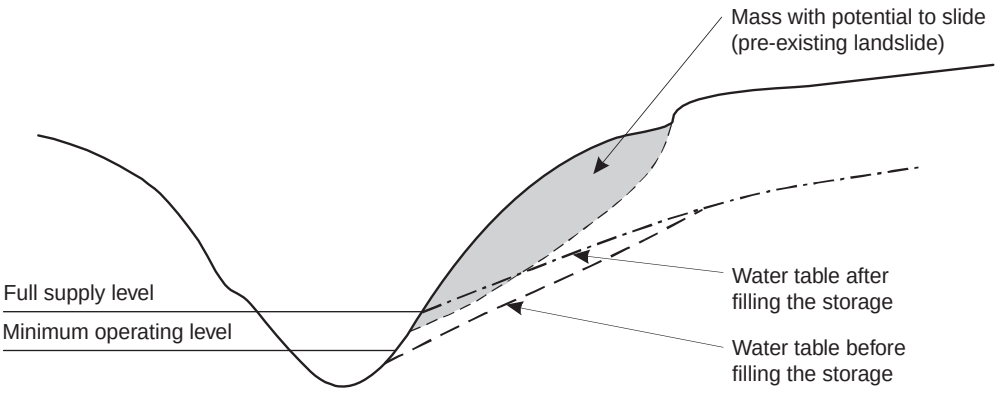


Figure 2.42 Effect of filling and operating a storage at the narrow base of a deep, steep-sided valley.

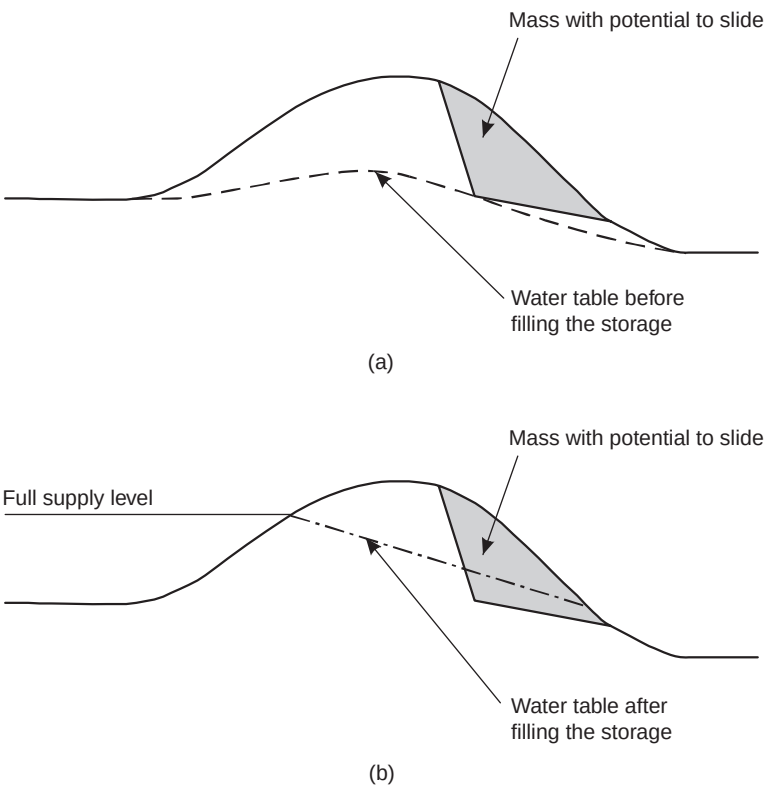


Figure 2.43 Effect of filling a storage on the stability of a low, narrow part of its rim, formed by jointed rock (a) Before filling the storage; (b) After filling the storage.

- Damage to the dam, or its foundations,
- Damage to or blocking of intake or outlet works,
- Damage to a spillway,
- Damage to a saddle dam or its foundations,
- Partial or complete blockage of the storage, where this is narrow, to form a landslide dam (or the perceived potential for this to occur). Flooding upstream and overtopping and eventual failure of this landslide dam are further possible consequences. Failures of such dams can be disastrous (Schuster and Costa 1986, and King *et al.*, 1987, 1989) or
- Generation of a large, destructive wave by rapid sliding into the reservoir, such as that which overtopped Vaiont Dam in Italy in 1963, and claimed 2600 lives (Hendron and Patton, 1985, Muller 1964, 1968, Kiersch 1965).

The consequences of landsliding on the outer parts of the storage rim may include:

- Constraints on operation of the storage,
- Damage to existing property or infrastructure,
- Damage to a saddle dam or its foundations (Cabrera, 1992) or
- In an extreme case, breaching of the rim, causing loss of storage and flooding of land.

It is clear that the possibility of landsliding is an important feasibility issue for storage projects. However, from examination of 145 reservoir landslides case histories, ICOLD (2002) has reported that:

- Only about 36% of the known reservoir landslides were recognised during the planning stage investigations,
- At least 75% of the known reservoir landslides were pre-existing dormant or occasionally active or active features and
- Such pre-existing slides cause the most problems for reservoir owners.

These figures indicate that the planning stage studies for some reservoir projects have been seriously ineffective. Recognising this, ICOLD (2002) has compiled guidelines on the identification and treatment of reservoir landslides, and management of the risks they create. The guidelines are aimed at both owners of existing projects and planners of future projects. They include 89 pages of text, figures and tables, 9 pages of references and 4 appendices. They cover all of the relevant issues, as follow:

1. International experience – case histories in Appendix A,
2. Management,
3. Geological and geotechnical investigations,
4. Slope assessment and analysis,
5. Impulse waves and valley blockage,
6. Risk management and risk mitigation,
7. Monitoring,
8. Operational requirements.

The authors commend the guidelines to readers. Assuming their availability and use by those involved with storage projects, the following text is limited to views on the timing of storage area stability studies and some strategies for managing instability issues should they become evident.

2.11.1 Vital slope stability questions for the feasibility and site selection stages

The authors consider it vital that the following questions be answered with confidence, by the end of Stage 2, Feasibility and Site Selection (see Chapter 4).

1. Which features (existing or proposed) and/or parts of the storage would be most vulnerable to landsliding, should this occur when the storage is filled or operated?
2. Which, if any, parts of the storage area rim show evidence of a) currently active landsliding, or b) old, dormant landslides which may become reactivated?
3. Which parts of the storage area rim may be susceptible to first time landsliding?
4. For each feature or area defined by the answers to 1 to 3 above, what is the assessed probability of landsliding, when the storage is filled and operated?
5. For each potential landslide, what is the likely volume, velocity and travel distance?
6. For each such feature or area, what are the assessed consequences of landsliding, under those conditions?
7. In the light of these assessments, what needs to be done, e.g. in the location and design of structures, in treatment of the ground, or in operation of the storage?

The sections which follow discuss when and how Questions 1 to 5 should be answered and include suggested strategies for action in respect to Questions 6 and 7.

2.11.1.1 *Most vulnerable existing or proposed project features, and parts of storage area? – Question 1*

This question can usually be largely answered in Stage 1 (Pre-feasibility), by examining contour plans showing existing infrastructure and buildings, the proposed storage in outline, and a preliminary layout of the proposed dam and associated works. From the lists in Section 2.11 above it is clear that generally, the project features most vulnerable to the effects of landsliding will be the dam, saddle dams, inlet and outlet works, spillways and roads. Also, topographically, the most vulnerable parts of the storage area will be the highest, steepest slopes within narrow sections (Figure 2.42) and steep, narrow ridges forming its rim (Figure 2.43). The exception may be existing landslides which are often not the steepest slopes.

2.11.1.2 *Currently active or old dormant landslides? – Questions 2 and 4 to 7*

Tentative answers to Question 2 should come also during Stage 1, from existing historical records, examination of air photos and from inspections from the air and on the ground. Currently active landslides and the scars or remnants of old, dormant landslides can usually be recognized and their plan boundaries delineated, during air-photo

interpretation (Section 5.2.2.2), and/or geomorphological mapping (Section 5.3) and geotechnical mapping (Section 5.4.2.2). Methods for their recognition and subsequent investigation are available in Rib and Liang (1978), Sowers and Royster (1978), Hunt (1984), Stapledon (1995) and Soeters and van Westen (1996).

In view of the ICOLD (2002) statistics quoted in Section 2.11, any pre-existing landslide feature found must carry a substantial risk of reactivation. The significance of a major pre-existing slide of either type in the rim of a storage area will depend on its size and location. Two possible situations are considered here.

Situation A – a slide which is judged to be sufficiently large and located so that its future movements could endanger a vulnerable existing feature or proposed project feature. For this situation the following approach might be followed, in Stage 2, Feasibility and Site Selection.

1. Do enough surface (and if necessary, subsurface) investigations to more accurately assess the size of the mass and its likely movements during storage operation.
2. If this confirms the size of the slide and that its predicted future movements are unacceptable, the following options should be considered.

   – Relocation of the existing feature, or proposed project feature, to a proven stable area or
   – Design of systems aimed at stabilizing the slide or limiting its displacements to acceptable velocities/amounts. Such systems are listed and discussed Section 9.3 of ICOLD (2002).

From experience, the authors believe that relocation is likely to be the better option in most cases.

Situation B – a slide which because of its apparent size and location (e.g. Figure 2.42) is judged to present a serious risk to the feasibility of the whole storage project. For this situation the following approach might be followed.

1. Do enough surface and subsurface investigations to prove whether or not the landslide is likely to become reactivated and is, when reactivated, large enough to present an unacceptable risk to the project feasibility.
2. If this shows that the slide is large enough and the risk unacceptable, then the following options would have to be considered:

   – Design of systems to stabilise it or
   – Abandonment of the proposed storage area.

3. If from the investigations in 1, it is concluded that the slide is not likely to be reactivated or that the reactivation will be slow moving and/or is not large enough for its activation to present an unacceptable risk to the project, it would still be prudent to confirm that conclusion by continuing to monitoring its behaviour through commissioning and early operation.

Stabilising systems for either situation A or B would have to be based on a geotechnical model of the landslide, capable of providing predictions of its future behaviour. As landslides are characterized by extreme variability in composition, structure and permeability (see Sections 2.10.2, 3.10.1.3 and 3.10.2.3), to produce such a model

would require a comprehensive and time-consuming investigation. Questions to be answered and investigation methods which can be used to answer them, are discussed in ICOLD (2002) and Stapledon (1995). However, it must be appreciated that at the start of such an investigation there could be no guarantee that its completion would answer all of the questions and allow the design of an adequate and economically feasible stabilizing system. This is made clear by ICOLD (2002) under Key Management Issues, as follows:

> "*Advanced techniques of field investigation, laboratory testing and stability analysis are now available. However, because of the complex nature of landslides and reservoir interaction, it is often difficult to define specific failure mechanisms and complete a meaningful stability analysis. This may leave substantial uncertainty in the determination of hazard and risk*".

2.11.1.3 *Areas where first-time landsliding may be induced (Questions 3 to 7)*

As pointed out in Section 2.10.1.2 first-time landslides are much more difficult to predict than reactivated landslides. They are often more important than existing landslides because they are more likely to travel rapidly.

a) *Rock slopes*. For rock slopes, a common type of first-time slide occurs where joints, bedding, or weak seams dip out of the slope and form all or part of a translational slab or wedge with kinematic freedom to move. The shaded wedges on Figures 2.41 and 2.43 are examples. Other possible first-time failure types include rockfalls, buckling and toppling. See also Section 2.10.3.5 and Section B on Figure 2.40 which shows the suspected potential first-time slide across the storage rim near Thomson Dam.

To assess the probability of a first-time slide from any rock slope requires (at least) knowledge of the pattern of defects within it. For a rock slope showing abundant outcrops or whose structure is otherwise well exposed (e.g. in exploratory trenches, shafts or tunnels) the plotted and analysed results of geotechnical mapping can indicate the probability that a kinematically unstable wedge or slab exists and its potential size. To assess the probability that such a kinematically unstable mass would actually fail, due to construction activities, operation of the storage or natural processes, would require substantial further site investigations. Some suggested questions to be answered in such investigations are set out in Appendix B of ICOLD (2002) and in Stapledon (1995). The assessment of the likelihood of such sliding is a technically complex matter and should be addressed by persons with expertise in rock mechanics and engineering geology as applied to large landslides.

Where such a suspect mass is located at or near a vulnerable project feature, the site investigation could form part of the site studies for that feature. However it would be preferable to look for an alternative, less suspect, site for the project feature and spend the time and money confirming its stability.

If the investigations show that the probability of landsliding is high and its consequences could affect the project feasibility, then the options for action would be:

- Design of stabilizing systems or
- Abandonment of the proposed storage area.

Table 2.7 Landslide mechanism classes and dominant source of brittleness (Glastonbury, 2002; Glastonbury and Fell, 2002c).

Brittle Toe Buttress	*Brittle Internal Deformation*	*Brittle Basal Rupture Surface*	*Brittle Lateral Rupture Surfaces*
Toe Buttress Compound	Bi-Planar Compound Curved Compound Toe-Buckling Translational	Rough Translational	Large Rock Glide
	⟵ Collapse/Slump Compound ⟶		
	⟵ Planar Translational ⟶		
	⟵ Irregular Compound ⟶		

b) *Soil slopes*. An apparently intact slope formed by soils would be suspect for first-time landsliding during filling or operation of a storage if the slope was known to have marginal stability and:

- It was close to areas also underlain by soils, but showing evidence of currently active or past landsliding,
- The underlying soils were currently well drained, at low moisture contents and the water table was low (well below the proposed storage levels),
- The soils were known to contain swelling clays or soluble materials such as halite or gypsum.

A large slope meeting any of those criteria would be best treated in the same way as a suspect rock slope, that is, avoided if possible during the location of any project feature. Should its potential size and location be such that its failure would endanger existing infrastructure or the project feasibility, then the options would be the same as those for a rock slope.

2.11.1.4 *What is the likely post failure velocity and travel distance?*

For landslides into the reservoir, it is large, rapid slides which are of most concern because they have the potential to cause waves which can overtop the dam, even breaching the dam.

Glastonbury (2002), Glastonbury and Fell (2002a, b, c) have developed a decision analysis framework for assessment of the post failure velocity of large natural rock slope failures based on the study of a large number of rapid and slow landslides. This shows that for a landslide to travel rapidly after failure there has to be a significant loss of shear strength on the surface of rupture and/or internally or on the lateral margins of the landslide or the factor of safety has to be maintained below 1.0 after failure by high groundwater pressures.

A slide having these characteristics is often considered as being "brittle" with the strength of a surface being brittle if there is a large loss of strength. Table 2.7 summarises the dominant sources of brittleness for the landslides studied by Glastonbury (2002). All the "rapid" slide cases examined in the database involved relatively high strength rock masses. Many rupture surfaces were pre-sheared, yet brittle collapse still occurred due to either brittleness on lateral margins or brittle internal deformation.

Table 2.8 and Figure 2.44 summarise the typical characteristics of the rapid landslides. The study also showed that many of the more obvious landslides associated

Table 2.8 Typical characteristics of large rapid natural rock landslides (Glastonbury 2002, Glastonbury and Fell 2002c).

		Translational slide class						*Compound slide classes*	
Characteristics	*Large rock glide*	*Rough translational*	*Planar translational*	*Toe buckling*	*Bi-planar compound*	*Curved compound*	*Toe buttress compound*	*Irregular compound*	*Rock collapse*
General geology	Sedimentary (esp. carbonate)	Anisotropic rock mass	Sedimentary (esp. carbonate)	Sedimentary	Variable – anisotropic	Folded sedimentary terrains	Anisotropic rock mass	Irregular – disturbed rock mass	Brittle cap rock over argillaceous strata
Slide Mass Characteristics	Very thick, (several hundreds of metres) generally intact	Some disaggregation	Intact	Intact	Often highly fractured	Structure to slope	Often highly fractured	Often partly fractured	
Rupture surface characteristics	Pre-sheared, bedding defined	Stress relief joints	Pre-sheared, bedding defined	Pre-sheared, bedding defined	Bi-planar, pre-sheared	Curved, pre-sheared	Upper surface pre-sheared	Irregular geometry	Rotational geometry in low strength rock
Rupture surface inclination	Very low (10–20°)	40°+	Typ 20–30°	Typ 30–40°	$\alpha_L = 20$–$30°$	$\alpha_L < 30°$	$\alpha_L = 10$–$25°$	$\alpha_{av} = 5$–$50°+$ rotational	Variable –
Rupture surface inclination v ϕ_b	$\alpha \approx \phi_b$	α typ 10–30° greater than ϕ_b	α typ within 5° of ϕ_b	α typ within 5° of ϕ_b	$\alpha_L \approx \phi_b$	α_L typ 2–3° less than ϕ_b	α_L typ 5° less than ϕ_b	α typ within 5° of ϕ_b	Variable
Rupture surface asperity influence	Minimal	High	Minimal	Minimal	Some	Minimal	Minimal	High	Likely some
Lateral margins	High influence	Moderate influence	Moderate influence	Low influence	Variable	Unknown	Unknown	Unknown	Moderate
Water influence	Unknown	Some	Common	Some	Some	Some	Limited	Unknown	Some
First time/ reactivated	First-time	Predominantly first-time	Both	First-time	Both	Both	Both	Both	First-time
Slope history (Dominant cause)	Glacial debuttressing	Fluvial and glacial	Fluvial erosion	Fluvial erosion	Glacial	Fluvial and glacial	Fluvial and glacial	Fluvial and glacial	Fluvial
Pre-collapse signs	Very limited	Limited	Limited	Limited	Unknown	Some	Common	Limited	Common

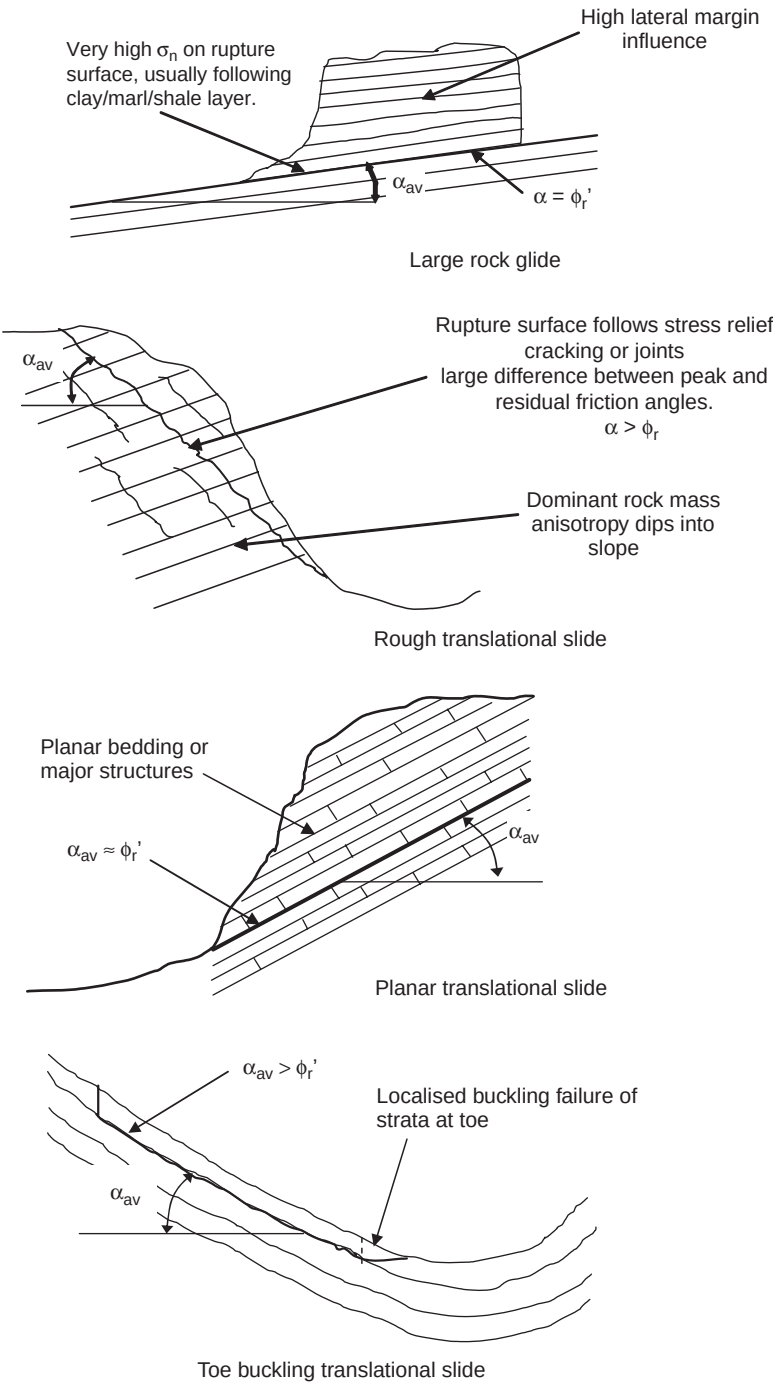
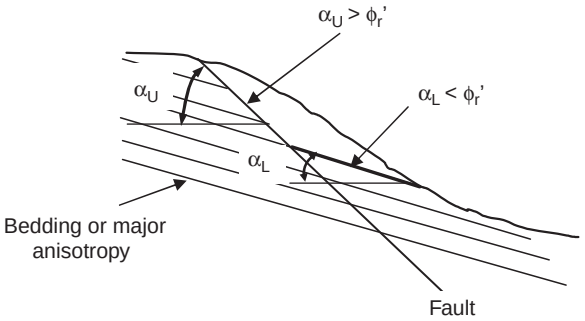
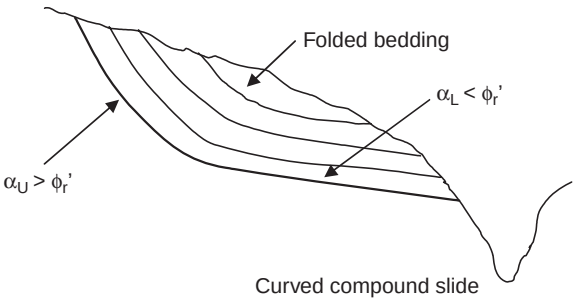

High lateral margin influence
Very high σ_n on rupture surface, usually following clay/marl/shale layer.
α_{av}
$\alpha = \phi_r'$
Large rock glide
α_{av}
Rupture surface follows stress relief cracking or joints large difference between peak and residual friction angles.
$\alpha > \phi_r$
Dominant rock mass anisotropy dips into slope
Rough translational slide
Planar bedding or major structures
$\alpha_{av} \approx \phi_r'$
α_{av}
Planar translational slide
$\alpha_{av} > \phi_r'$
Localised buckling failure of strata at toe
α_{av}
Toe buckling translational slide

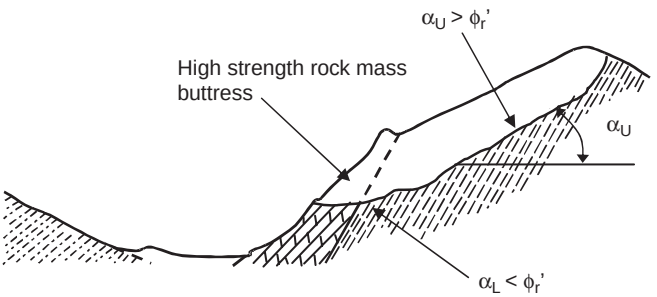
Figure 2.44 Summary of features of large rapid rock landslides (Glastonbury 2002, Glastonbury and Fell 2002c).



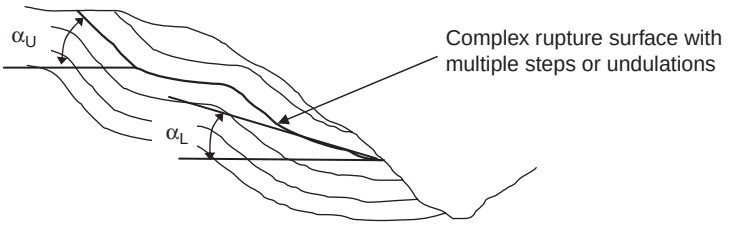
Bi-Planar compound slide



Curved compound slide



Toe buttress compound slide



Irregular compound slide

Figure 2.44 Continued.

with reservoirs which have been studied and monitored are likely to be slow moving because they are active or reactivated slides with little likelihood of brittle failure.

2.11.1.5 *What is the size of impulse waves which may be created?*

The magnitude of the impulse waves generated by rapid landslides moving into a reservoir can be predicted using the methods by Panizzo *et al.* (2005), Heller (2008) and Heller *et al.* (2009). The size of impulse waves depend on many factors of which the key ones are:

- The volume of displaced water, which depends primarily on the thickness of the landslide.
- The velocity of the landslide as it enters the water.
- Whether the landslide is relatively intact ("solid body") or broken up ("granulate") as it enters the water.
- Whether the resulting waves are travelling confined within a valley (so-called 2-dimensional) or whether they are free to radiate (so called 3-dimensional).

2.12 WATERTIGHTNESS OF STORAGES

Absolute watertightness is unlikely to be achieved in most natural storages. However it is usually desirable that the rates of any leakage are minimized, for any of the following reasons:

- The cost of water lost can be unacceptably large e.g. in arid areas, or in off stream storages fed by pumping,
- In some situations leakage may cause raising of water tables, development of swamps, or flooding, in areas adjacent or downstream.

In a few cases leakage has caused instability of slopes on the outer edge of the storage rim (see Section 2.11). The watertightness of a storage basin will depend upon:

- The permeability of the underlying rock or soil mass,
- The permeability of the rock or soil mass surrounding it,
- The groundwater situations in those masses and
- The lengths and gradients of potential leakage paths.

Masses of soluble rocks (carbonates and evaporites, see Sections 3.7 and 3.8) are often cavernous and have permeabilities many orders higher than those of non-soluble rocks. This discussion on storages will therefore consider non-soluble rocks separately from those on soluble rocks.

2.12.1 Models for watertightness of storages in many areas of non-soluble rocks

Most non-soluble rock substances are effectively impervious, and rock masses formed by them are only permeable if sufficient numbers of interconnected open joints are present. As pointed out in Section 2.5.2, joints generally

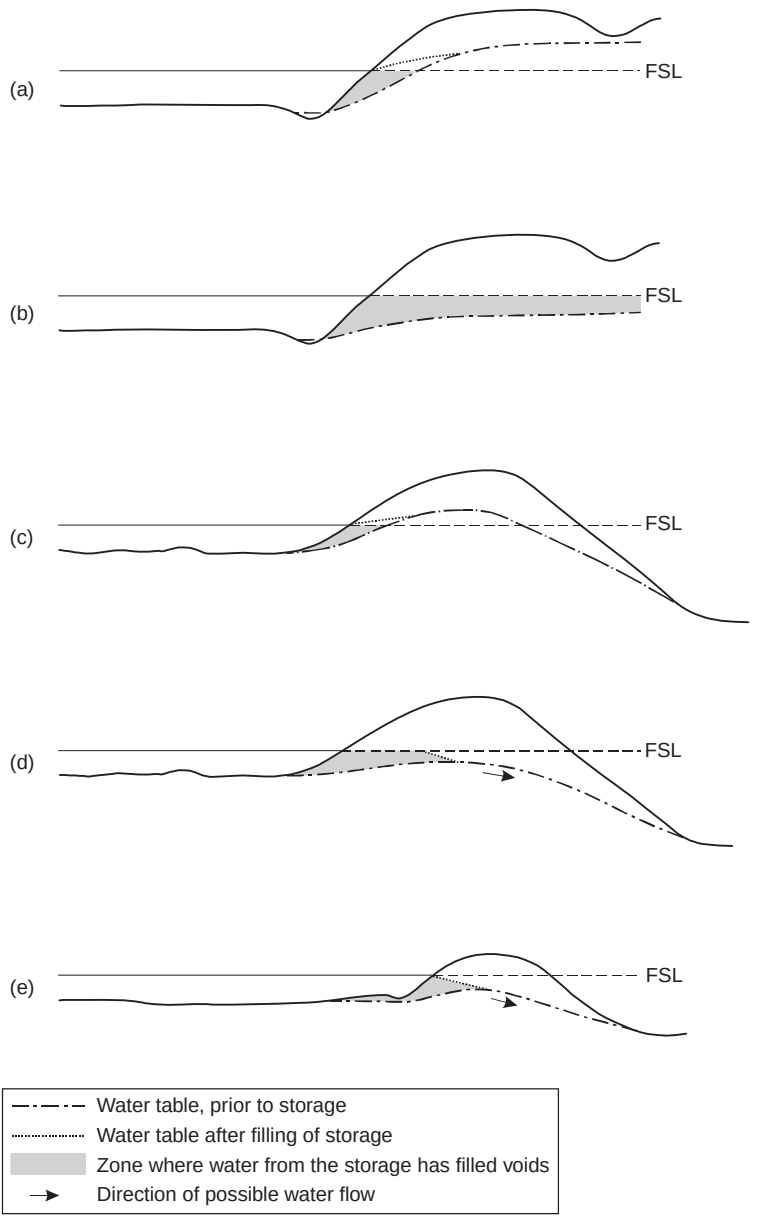


Figure 2.45 Storages formed by non-soluble rocks – watertightness models.

become more tightly closed with increasing depth, so most masses of non-soluble rock will become less permeable with increasing depth. Also, their effective porosities (or storage capacities) decrease with increasing depth and are usually very low (i.e. 10^{-3} to 10^{-6}). The diagrams on Figure 2.45 show the effects of creating

storages in such areas of non-soluble rock, in five relatively common topographic and groundwater situations. In the discussion of each it has been assumed that:

(i) The rock becomes less permeable with increasing depth,
(ii) Across the floor of the storage area, the water table lies close to the ground surface,
(iii) The water tables shown represent end of dry season conditions,
(iv) Groundwater pressures measured below the water table as shown will all confirm it as the piezometric surface and
(v) Flow beneath the high ground or ridges forming the rims will be essentially laterally to or from the storage, unlike the isotropic material case, proposed by Hubbert (1940).

The perimeter of any particular storage area may include sections topographically similar to any or all of Models a/b, c/d or e.

Model (a). In this case the rainfall is high and the storage is in an entrenched valley underlain by rock and surrounded by wide areas of high ground underlain by rock. The rock in the valley floor may be partly or wholly concealed by soils of alluvial or glacial origin. Beneath the high ground, the water table rises above the proposed FSL. There is no potential for leakage. The storage should be essentially watertight, and will be fed partly by groundwater. As the storage fills, a very small amount of water will fill the voids in the adjacent rock mass (shaded). When the storage is full, the water table beneath the valley side will have risen slightly and will meet it at the FSL (see dotted line).

There are many storages fitting this model in non-soluble rock areas with high rainfall.

Model (b). The topographic and geological situations are similar to those for Model (a), but the water table lies below the proposed FSL. There may be some bodies of perched water, above this true water table. The reasons for the water table being so much lower than for Model (a), may include any or all of the following:

1. The climate is drier,
2. The surface runoff is greater,
3. The rock is more permeable and drains more rapidly towards the valleys.

In this situation there is potential for leakage, but leakage paths would be so long and at such low gradients, that loss from the storage would probably be negligible. The shaded area indicates where small amounts of the storage water will enter, fill voids and cause the water table to rise locally.

It is likely that this model represents many storages in non-soluble rock areas with low to moderate rainfall.

Model (c). This storage is elevated well above an adjacent valley and separated from it by a high, broad, ridge. The water table within the ridge rises above the proposed FSL. There is no potential for leakage from the storage to the valley, so the storage should be essentially watertight. A small amount of storage water will fill voids above the water table in the adjacent valley side (shaded) and the water table will rise slightly and meet this valley side at FSL.

Model (d). The topography and geology are the same as Model (c), but the water table does not rise above the proposed FSL. The reasons for this may include any or all of the following:

– The climate is drier,
– The surface runoff is greater,
– The rock is more permeable and drains more rapidly towards the valleys.

There is potential for leakage. However, because of the high rock cover and long potential leakage path, significant leakage would be unlikely. A small amount of storage water will fill voids in the rock above the water table in the adjacent valley side (shaded) and should cause some rise in the water table.

Model (e). The storage is elevated, as for Models (c) and (d), but the ridge forming its rim is low and narrow. As a result of this, the rock beneath the ridge and below FSL is likely to be affected by de-stressing, and hence more permeable than that at Models (c) and (d). If the rainfall is high, as in the case shown, the water table may rise slightly beneath the ridge, but would be unlikely to rise above the proposed FSL, as at Model (c). When the storage is filled, seepage from it will cause the water table to rise, to at least the position of the dotted line. There is potential for leakage from the storage across to the lower valley (see arrow).

2.12.2 Watertightness of storage areas formed by soluble rocks

As discussed in Chapter 3, Section 3.7 Carbonates and Section 3.8 Evaporites, areas underlain by these soluble rocks often exhibit karst topography and may be cavernous to great depths. In carbonate rocks, depths of more than 300 m has been proven at Attaturk Dam (Riemer *et al.*, 1997) and about 600 m at Lar Dam (Salambier *et al.*, 1998). Ruquing (1981) records cavities developed to about 1200 m below the Yilihe River in China.

The evaporites are more soluble than the carbonates and pose much more difficult problems in dam engineering. The factors affecting watertightness of storages are essentially the same for both, but there have been many more dams built in carbonate areas. Therefore, the following discussion will deal with principles and experiences from carbonate rock areas.

The carbonate rocks can occur in beds ranging in thickness from a metre or so up to several hundred metres, e.g. see Figure 3.29, a cross section at Lar Dam. They commonly occur together with beds of non-soluble rocks, in uplifted sequences which have been folded, jointed and displaced and locally crushed by faulting. Such complex rock masses will have been affected by tectonic movements and dissolution at various times during their long geological history. In the more recent episodes they have been selectively dissolved and eroded, both at the surface, producing deep river valleys and underground producing interconnected caverns and complex flow paths.

There can be many types of direct connections between surface and underground streams. For example, during the wet season a surface river may flow into a tunnel or shaft, flow underground for a kilometer or more and emerge as a spring, either high on the side of another valley or lower down, close to another surface river. This flow pattern might be quite different during the dry season.

A further complication at some sites is blanketing of the valley floor by either residual soils, or sediments deposited by rivers or lakes. Obvious sinkholes or subtle depressions on these valley floors indicate that material is or has been eroding or collapsing into cavities in the underlying karst. The river and storage area at Lar are underlain by up to 600 m of lake sediments and before impounding the water table was about 200 m below the river bed. Such rivers and storages are referred to as "supported" or "hanging".

As a result of this degree of complexity, it is not possible to present a set of models for storages in karst areas, equivalent to those on Figure 2.45. Also, it can rarely be safely assumed that a karstic rock mass will become less permeable at depth. This means that assumption i) on which the Figure 2.45 models are based, does not usually apply. It might appear that the right abutment ridge at Khao Laem Dam fits Model (b) (see Section 3.7.2). The ridge was formed by cavernous limestone with a dry season water table above the valley floor but below the proposed FSL. However, if the dry season water level in Khao Laem ridge had been above the proposed FSL, it would not have been safe to assume a Model (a) (i.e. watertight) situation, unless it could be proved that the limestone:

– Became free of interconnecting cavities, and effectively impermeable, at depth or
– Was not within a bed which emerged at a lower level, either downstream or in another valley.

A classification of karst river valleys has been compiled by Ruquing (1981) using experience gained during engineering works in the Yunnan and Guizhow Provinces of southwest china. The classification includes notes on the potential of each valley type for the storage of water. Ruquing (1981) recognizes 5 categories of 10 types, but notes that the classification is far from complete.

2.12.3 Features which may form local zones of high leakage, from any storage area

Some proposed storage areas underlain mainly by non-soluble rocks may contain localised individual features of very high permeability, which could potentially act as drains. Such features may include:

– Buried channels, alluvial or glacial (see Sections 3.9.1 and 3.12.1),
– Lava flows with clinker (Section 3.2.2.1),
– Lava tubes (see Section 3.2.2.1),
– Some open-jointed fault zones (see Section 2.3.2),
– Abandoned underground mine workings.

It is probable that such features would rarely cause significant leakage from storages fitting Models (a) or (b), because leakage paths would be too long and gradients too low. However they could form effective drains, if located within or through the outer ridges in Models (c), (d) and (e).

Any of these features could occur also within a storage area underlain by soluble rocks and might influence its watertightness. It is also possible that an isolated bed of cavernous, soluble rock might occur within a storage area formed almost entirely by non- soluble rock and influence its watertightness.

2.12.4 Watertightness of storages underlain by soils

Many dams and weirs are built in the valleys of large rivers and the reservoir watertightness is controlled by the alluvial or glacial soils. Often the water tables are relatively high and graded towards the river, so the issue is essentially one of the limiting seepage under and around the ends of the embankment as discussed in Chapter 10. Some storages have to rely on liner systems for watertightness if the underlying soils are very permeable, deep and/or the water table is low.

2.12.5 Assessment of watertightness

During the planning stages of the project (Stages 1 to 3 on Table 4.4) the following questions should be addressed and answered:

1. Will the proposed storage area be essentially watertight?
2. If not, where are the areas of potential leakage?
3. What is the estimated total leakage rate?
4. What are the consequences of this leakage?
5. What, if any, treatment is needed?

The amount and types of investigation needed to answer these questions will depend largely on whether the storage area is formed in essentially non-soluble rocks or largely on soluble rocks or on soils. This "overall geological situation" will usually be evident from examination of existing regional geological maps. If these are not available, new mapping will be needed. It is important also that the existing maps or new mapping provide an understanding of the geological (and modern) history of the area.

2.12.5.1 *Storages in non-soluble rock areas – assessment of watertightness*

It is usually possible to get preliminary answers to Questions 1 to 3 using existing contour plans, regional geological maps, air photos and climatic and groundwater records. The perimeter of the proposed storage area is divided into a number of watertightness regimes, based on differences in topography and assumed geology and groundwater situations. A judgement on the probable watertightness of each regime is made by comparison with the models on Figure 2.45. Regimes which appear to fit Models (a), (b) and (c) are assumed to be watertight. For regimes which appear to fit Models (d) or (e), ranges of possible leakage rates through each can be obtained by simple calculations using wide ranges of possible rock mass permeabilities and assumed flow paths, cross sectional areas and gradients. Construction of realistic flow nets is unlikely to be possible, or necessary.

If the geological and modern history suggests that high permeability features as listed in Section 2.12.3 may occur, but none are shown on the geological maps, they can be looked for on the air photos. They are often indicated by areas of anomalous vegetation. If such a feature is recognised and confirmed by ground inspection, ranges of possible leakage rates through it can be found be similar simple calculations.

The results of the above preliminary assessment are considered in relation to the consequences of leakage, which will usually be known. If the largest calculated leakage

rates are shown to be of no consequence and the geological and groundwater data known to be reliable, then the watertightness studies could be considered complete, with no further action needed.

If some of the calculated leakages are seen to be significant and there are doubts about the reliability or sufficiency of the geological or groundwater data, further field and office studies would be required. The field studies may include any or all of the following:

- Further regional geological mapping,
- Plotting the positions and levels of springs or seepages, and measuring flow rates,
- Plotting the positions and levels of existing water wells or bores and recording of water levels in them,
- Detailed studies of any recognized high permeability features and of parts of the storage rim which are suspect for leakage because of, for example, their lack of rock exposures, low and narrow shapes, suspected low water tables and steep potential leakage gradients. Such studies have typically included detailed surface mapping, excavation and mapping of trenches and drilling and permeability testing of cored boreholes. The positions of water tables may be judged during the progress of drilling, using the DVD method (see Chapter 5), but would have to be confirmed by piezometers.

The results of the above, presented in plans, sections and diagrams, would be used for new calculations of potential leakage rates. It is unlikely that the structure-related permeability of any jointed rock zones would be sufficiently well known to justify more than the simplest flow nets.

In cases where the calculated rates are deemed to be unacceptable, consideration might be given to alternative storage sites or to measures such as blanketing or construction of a grout curtain or slurry trench (see Sections 2.12.6 and 10.4). Further investigation would be needed before adoption of any of these.

2.12.5.2 *Storages in soluble rock areas – assessment of watertightness*

From the discussion in Section 2.12.2 it is clear that the flow paths developed in a soluble rock area may have developed as results of:

- Past, and in some cases presently active, tectonic processes,
- Dissolution and erosion by surface and underground waters, at times when the geological model and topography were quite different to those of today and
- Dissolution and erosion by surface and underground waters, in the present day geology and topography.

The keys to watertightness assessment therefore lie in:

- Knowing the regional topography and geological picture,
- Understanding the geological history and
- Understanding the flow pattern of the present day surface and underground waters and their relationship to the geological structure and history.

Accordingly, the initial activities would usually include:

- Using existing contour and regional geological plans (supplemented where necessary by new mapping) to compile cross sections and/or diagrams providing a 3-dimensional picture of the geological structure beneath and surrounding the storage area,
- Compiling a table and diagrams explaining the assessed geological history and
- Hydrological studies, including recording wet and dry season flow rates of surface streams and springs, water levels in wells and boreholes.

The likely overall watertightness, and likely areas and models for leakage, would be assessed from the results of the above. Consideration of the range of models described by Ruquing (1981) and of experiences at the dams listed in Tables 3.3 and 3.4 should be helpful. When likely areas of leakage are recognised, they can be investigated further using any of the following methods:

- Detailed geological surface mapping, plus trenching or test pitting.
- Mapping of caves,
- Geophysical methods, which might include microgravity, resistivity or ground probing radar (from the surface or between boreholes),
- Core- and non-core drilling, preferably with continuous recording of penetration rates and pressures (to assist in recognition of cavities),
- Permeability testing, by the Lugeon method or (preferably) by pump out tests;
- Installation and monitoring of piezometers,
- Further hydrological studies, including water analyses, tritium dating, tracer studies (Gasper and Onescu, 1972, Habic, 1976, and Eriksson, 1985), temperature measurements (Fuxing and Changhua, 1981), spring blocking studies (Fuxing and Changhua, 1981) and pulse analysis (Williams, 1977). Breznik (1998, Page 55) describes the use of "geobombs" or large explosive devices, to locate the positions of underground channels unreachable by speleologists.

2.12.5.3 *Storages formed in soils – assessment of watertightness*

The watertightness of a storage formed by a soil mass will depend on the permeabilities and resistance to piping erosion of its component soils and their boundaries and configurations in relation to potential leakage paths. Understanding the site history and the soil origin type or types can provide clues to the likely nature and orientations of soil boundaries, and to the presence of defects and other features likely to affect permeability (see Sections 3.9 to 3.13). Assessment should therefore begin with examination of existing geological maps and reports and making detailed observations and mapping of soils exposed in erosion gullies, river banks or excavations. Other initial activities would include simple tests for dispersive soils, collection of published data on groundwater, plotting the position of bores, measuring water levels and plotting water level contours.

A preliminary geotechnical model for the proposed storage floor and rim or a model for each of several regimes can be developed from the above activities. Simple calculations using assumed parameters and possible flow paths should indicate whether a storage is likely to be feasible, with or without lining or extensive treatment or both.

More detailed studies, planned to answer questions raised from the results of the above, might include the following:

- Excavation of trenches or test pits and detailed logging of exposed faces after cleanup by hand tools and compressed air,
- Drilling and undisturbed sampling of boreholes, geological and engineering logging of the samples, testing them for moisture contents and grading and if appropriate, Atterberg Limits and erodibility,
- Installation and monitoring of piezometers, located at positions indicated by the logs and moisture profiles,
- Permeability tests in boreholes,
- Excavation and conducting infiltrometer tests in uniformly shaped pits, (1) after cleanup, logging and photographing the floors and (2) after remoulding the floor soil to a standard degree and depth„
- Construction of trial ponds or tanks and using these for large-scale permeability tests, and to test the applicability of various soil treatments and linings.

2.12.6 Methods used to prevent or limit leakages from storages

Localised features shown to cause unacceptable leakage can be treated by cement grout curtains, concrete diaphragm walls or mining and backfilling with concrete. In karst carbonates it is common to carry out extensive grouting and excavation/backfilling from adits into the abutments. These may extend considerable distances.

A clay blanket together with a soil-cement grout curtain was successful in reducing leakage through weathered gneiss forming a critical part of the rim of Lajes Reservoir, in Brazil (Cabrera, 1991).

Breznik (1998, Page 131) reports that sealing of rocky slopes in karst areas, by shot-creting and plugging of individual caves with concrete, has generally been successful in preventing leakage. These methods may be applicable also for lava tubes, lava flows or old mine workings.

Breznik (1998, Pages 126–130) describes how sediments covering the floor of the Popovo "hanging" storage in Herzegovina were treated locally by blanketing with low permeability soils, covered in parts by plastic sheeting. "Chimney" pipes were installed to prevent disruption of the blanket by air expelled from cavities when the water table rose. The treatment was successful at this site but similar treatments at four other sites were not.

As noted in Section 3.7.2.1, shotcreting, concreting and a geomembrane blanket have been recommended for the Lar storage area adjacent to the dam. Riemer (2003) has advised that these works have not been carried out.

Breznik (1998) notes that localized areas of sinkholes near the margins of some storage areas have been isolated from the storages by construction of low embankment dams.

Chengjie and Shuyong (1981) describe the effectiveness of an 8 m high concrete masonry dam which was used to isolate an area of sinkholes from the Baihua hydroelectric storage in Southwest China.

Chapter 3

Geotechnical questions associated with various geological environments

As explained in Chapter 2 and later in Chapter 4, site investigations for a dam need to be undertaken with a good understanding of the local and regional geological environment and the investigations should be aimed at answering all questions known to be of relevance to dam construction and operation in that environment. This chapter discusses twelve common geological environments in which dams have been built and derives check lists of geotechnical questions of specific relevance to each.

It is important that readers appreciate the limitations of these generalisations and check lists. The lists refer simply to features that might be present because they have been found during construction at many other sites in similar environments and because geological reasoning suggests that they could be present. At any particular site the actual geological conditions found will have been developed as a result of many geological processes acting at different times over vast periods of geological time. If some of these processes have been very different from those assumed in the "general" case, then some or even all of the generalisations may not be valid at that particular site.

For further accounts of regional and local geological environments, with derived predictions of site conditions, see Fookes (1997) and Fookes *et al.* (2000).

3.1 GRANITIC ROCKS

Included under this heading are granite and other medium or coarse grained igneous rocks. Most rocks of these types have been formed by the cooling and solidification of large masses of viscous magma, generally at depths of greater than 5 km below the ground surface.

3.1.1 Fresh granitic rocks, properties and uses

In unweathered (fresh) exposures, granitic rocks are usually highly durable, strong to extremely strong (substances) and contain very widely spaced (greater than 2 m) tectonic joints in a roughly rectangular pattern (Figure 3.1). Many of these joints are wholly or partly healed by thin veins of quartz, or quartz/felspar mixtures. Sheet joints are common but, as they are almost parallel to the ground surface, they may be difficult to detect during surface mapping.

Fresh granitic rocks are commonly quarried for rip-rap, rockfill and concrete aggregates, but mica-rich granites may be unsuitable for use as fine aggregates in concrete due to excessive amounts of fine, platy particles in the crushed products.



Figure 3.1 Granitic cliff showing 3 sets of joints approximately at right angles to one another.

3.1.2 Weathered granitic rocks, properties, uses and profiles

Chemical weathering of granitic substances usually causes cracking at the grain boundaries and decomposition of the felspars and ferromagnesian (dark) minerals, leaving quartz grains essentially unaffected.

Table 2.6 is a practical descriptive classification scheme for weathered granitic rocks. When extremely weathered (i.e. soil properties) most granitic materials are silty or clayey fine gravels or sands (GM, GC, SM or SC). *In situ* these materials are usually dense to very dense, but in some tropically weathered areas, where quartz has been partly or wholly removed, they are more clay rich and of low density. Somerford *et al*. (1991) and Bradbury (1990) describe low density, extremely weathered granitic materials at Harris Dam, Western Australia.

The extremely weathered materials often make good core or earth fill materials and where the parent rock is very coarse grained the resulting gravels can make good quality road sub-base for sealed roads or base course for haul roads.

The silty nature of some extremely weathered granitic rocks often causes them to be highly erodible, when exposed in excavation and when used in fills. At Cardinia earth and rockfill dam near Melbourne extremely weathered granite is dispersive and where exposed in the storage area shoreline has required blanketing with rockfill to prevent erosion and subsequent water turbidity problems.

Lumb (1982) describes engineering properties of granitic rocks in various weathered conditions.

Typical weathered profiles in granitic rock masses are shown on Figures 2.17 to 2.22. The chemical weathering is initiated at and proceeds from the ground surface and from sheet-joints, tectonic joints and faults, causing the roughly rectangular joint-blocks to become smaller, rounded and separated by weathered materials. Thus the profile grades usually from residual granitic soil near the surface to fresh rock at depth,

with varying amounts of residual "boulders" of fresh or partly weathered rock occurring at any level. Fresh outcrops or large fresh boulders at the ground surface may or may not be underlain by fresh rock. It is not uncommon to find that weathering has occurred beneath such outcrops, along sheet joints, gently dipping tectonic joints, or within previously altered granitic rock (Figure 2.22). Understanding of this potential for variability in weathered granite profiles is important not only for dam foundations, but also when planning and operating either a quarry for rockfill, rip-rap, filters or aggregate or a borrow pit for earth fill or core materials.

3.1.3 Stability of slopes in granitic rocks

Active landsliding, or evidence of past landsliding, is relatively common in steep country underlain by weathered granitic rocks, particularly in areas with high rainfall. Brand (1984) describes the widespread occurrence of natural and man-induced landslides in weathered granitic rocks in Hong Kong.

Although some landslides in weathered granitic rocks no doubt occur by failure through the fabric of extremely weathered material, it is probable that in many landslides failure occurs wholly or partly along relict joints or other defects (see Chapter 2, Section 2.10 and Figure 2.28).

The landsliding at Tooma and Wungong dams (Sections 2.10.3.2 and 2.10.3.3) occurred along localised weathered zones along pre-existing defects and past movements had occurred well into rock masses which were dominantly slightly weathered.

3.1.4 Granitic rocks: check list

- Concealed sheet joints?
- Fresh rock outcrop; does it extend down into fresh rock?
- Chemically altered zone(s)?
- Fresh granite "boulders" within extremely weathered materials?
- Extremely weathered materials, suitable for impervious core? Road pavements? Highly erodible? Low density *in situ*?
- Past landsliding? Stability of extremely weathered materials in cuts?

3.2 VOLCANIC ROCKS (INTRUSIVE AND FLOW)

The common rocks in this group range from basalt (basic) through andesite, dacite, trachyte, to rhyolite (acidic). Basalt is the most common. All are formed from molten magma and are very fine grained, usually very strong to extremely strong when fresh. In this fresh condition the rocks generally are also very durable and are used commonly as sources of materials for filters, concrete aggregates, rockfill and road base courses. However volcanic rocks, particularly basalts and andesites, often show subtle alteration effects, which in some cases render them unsuitable for some or all of these purposes. This matter will be discussed further in Section 3.2.3. Also most volcanic rocks have initially contained some glass. In rocks of Mesozoic age and older the glass has usually "devitrified" or crystallized. However, in rocks of Tertiary and younger age the glass is usually still present today and, if the rock is used as concrete aggregate,

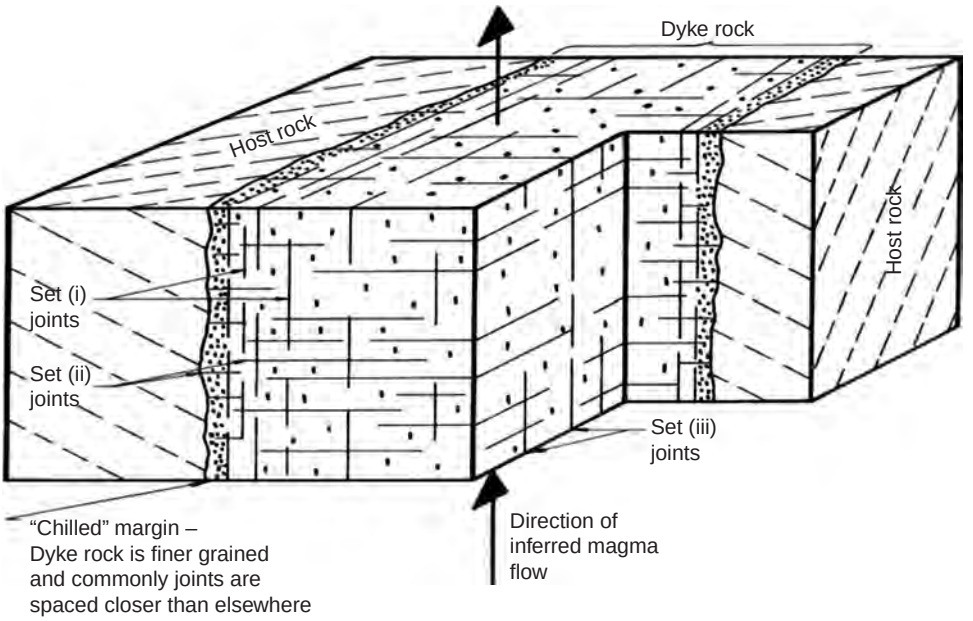


Figure 3.2 Some features commonly seen in dyke of intrusive rock.

it may react with alkalis in the cement and cause the concrete to deteriorate (see also Section 3.2.6).

The shape and other field characteristics of a body of volcanic rock depends upon the circumstances in which it solidified, i.e. as a plug, dyke, sill or flow.

3.2.1 Intrusive plugs, dykes and sills

In these types of bodies the magma has been confined within other rocks (or soils), and has flowed against them and eventually solidified against them. As a result of this mode of formation, any of the following characteristics shown in Figure 3.2 are commonly seen:

(a) The host-rock (or soil) close to the contacts may be stronger and more durable than elsewhere due to being subjected to very high temperatures.
(b) The intrusive rock has "chilled" margins, i.e. it is extremely fine-grained or even glassy, close to its contacts, due to a faster rate of cooling than in the interior of the mass.
(c) The intrusive rock has developed a "planar" foliation parallel to its contacts and, within this, a lineation or linear arrangement of mineral grains, parallel to the direction of flow during intrusion.
(d) Joints in the intrusive body occur in at least 3 sets, as shown on Figure 3.2. Set (i) joints are parallel to the contacts (and to the foliation). Set (ii) joints are normal to the lineation i.e. to the direction of magma flow and also to the contacts. Set (iii)

joints are normal to the contacts and parallel to the lineation direction. The joints in all sets commonly show extension characteristics (i.e. rough or plumose surfaces) and are either slightly open or infilled with secondary minerals including calcite and zeolite minerals. It can be inferred that shrinkage was an important factor in their formation, although extension during viscous flow seems a likely initiating factor for Set (ii).

It is sometimes found that both the host and intrusive rock are sheared or crushed, along and near the contact zone. In some cases this appears to be as a result of viscous drag, but more generally tectonically induced movements after solidification appear to be the likely cause.

In some cases, open joints in the body and contact zones may render the intrusive mass highly permeable. If continuous, such permeable masses represent potential leakage zones beneath dams or from storages.

3.2.2 Flows

Lava "flows" are bodies which have been formed by molten lava which has been extruded at the ground surface or the sea floor and has flowed over a pre-existing surface of rock, soil or sediment.

3.2.2.1 *Flows on land*

Structures developed in lava flows on land are described in some detail in Hess and Poldervaart (1967), Francis (1976) and Bell (1983a). The flows move forward in a manner similar to that of a caterpillar tractor, as shown in Figure 3.3. The lava near the exposed, upper surface of the flow usually develops many small holes or vesicules, formed by bubbles of expanding gases becoming trapped as the magma solidifies. The upper surfaces of some flows develop an extremely rough, fragmented structure, comprising sharp, irregular fragments up to 150 mm across, termed clinker. Bell reports that clinker layers several million years old and buried at 500 m to 1000 m depths in Hawaii show little or no sign of compaction and are highly permeable. As the flow (Figure 3.3) moves forward the clinker-covered surface and vesicular layer are carried forward, deposited over the front and eventually buried beneath the flow. Thus the solidified flow comprises an inner layer of massive rock sandwiched between two layers of clinker and vesicular rock which are highly permeable.

Other lava flows develop a hummocky and sometimes twisted ropy structure at their upper surfaces, due to viscous drag on the surface crust while this is still plastic.

Where successive lava flows have occurred to form a continuous layered sequence, the individual flows and their boundaries may be distinguishable by the following:

- The development of a weathered or soil profile on the upper surface of a flow which was exposed to weathering for some time before the next flow occurred.
- The presence of chilled and vesicular zones, or clinkered, brecciated or ropy zones near flow boundaries.

At Foz do Areia dam in Brazil, breccia (clinker) zones at the boundaries of basalt flows were locally weathered and generally highly permeable (Figure 3.4). Treatment of these zones by excavation, dental concrete and grouting is described by Pinto *et al.* (1985a).

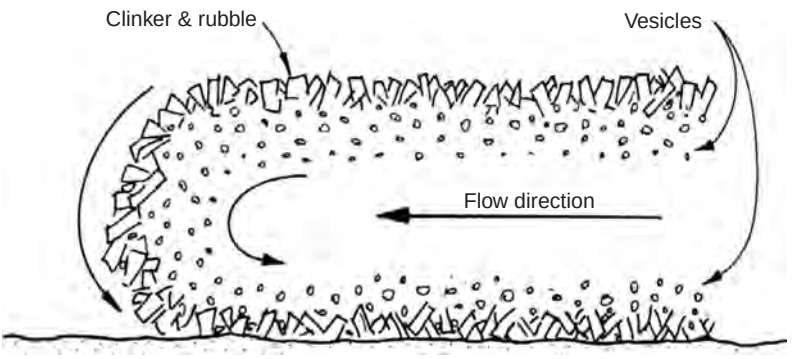


Figure 3.3 Diagrammatic longitudinal section through flowing lava showing the "caterpillar track" mechanism which results in upper and lower layers of vesicular lava and in some cases clinker or breccia.

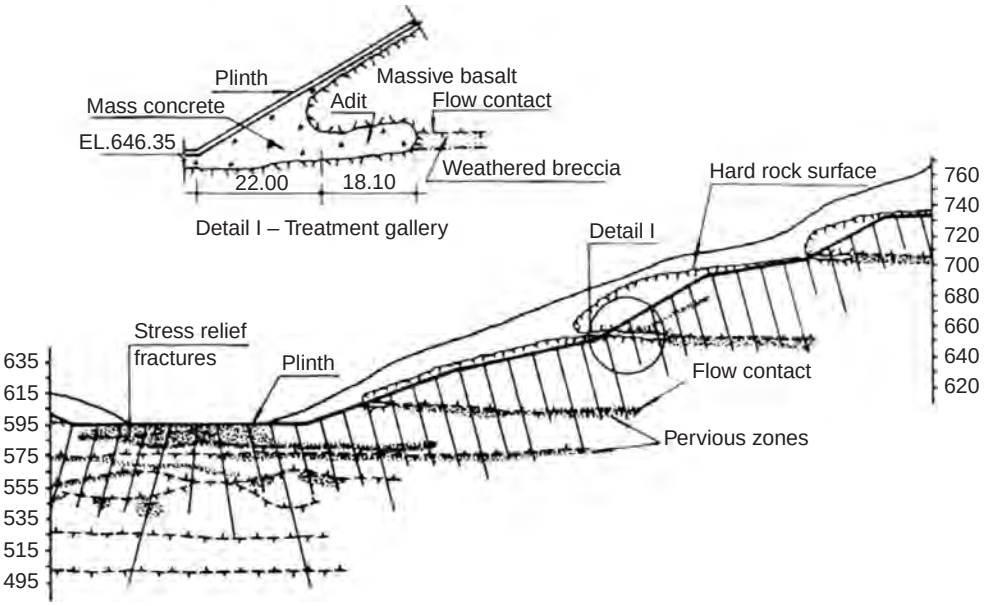


Figure 3.4 Geological section along Plinth (right bank) at Foz Do Areia Dam (Pinto *et al*., 1985a).

It is common for lava flows to be interbedded with pyroclastic materials (ash and lava fragments) or tuff and agglomerate in rock sequences derived from volcanic explosions. Also flows may occur interbedded with alluvial or other sediments.

Thick lava flows sometimes contain "lava tunnels", which are circular, ovoid or lenticular in cross section, up to 20 m across and some kilometres in length. These are formed when the supply of lava to the flow is exhausted, and the internal lava "stream" (Figure 3.5) drains away. Figure 3.6 shows a small lava tunnel exposed

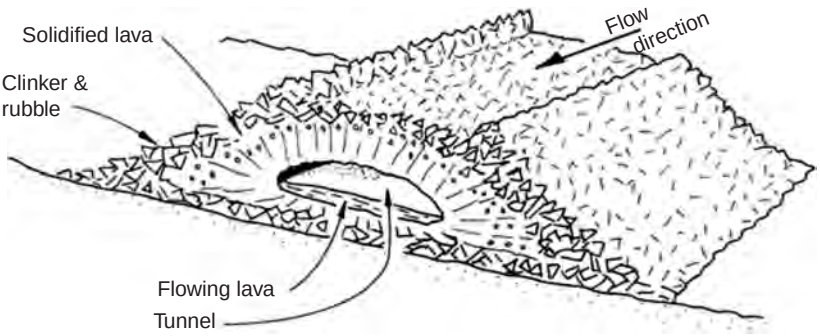


Figure 3.5 Perspective view of, and cross section through a lava flow, showing a lava tunnel developed when the lava flows forward faster than the supply.



Figure 3.6 Lava tunnel in basalt, Hoppers Crossing, Victoria, Australia.



Figure 3.7 Columnar joint pattern in rhyolite, and younger intrusive dyke. High Island Dam, Hong Kong.

in basalt forming the wall of a 30 m diameter shaft at Hoppers Crossing, Victoria, Australia.

Most lava flows show a hexagonal columnar joint pattern, with the columns being interrupted by near-planar or saucer-shaped cross joints. These joints have developed as a result of shrinkage on cooling and are usually either slightly open or filled with secondary minerals or alteration products. Figure 3.7 shows columnar-jointed rhyolite exposed during construction at High Island dam, in Hong Kong. The rhyolite has been intruded by a younger, darker rock forming a dyke up to 1.5 m thick.

In some flows there are closely spaced joints parallel and near to the margins, apparently caused by shearing associated with viscous flow. Where columnar joints in lava flows are open, e.g. due to mechanical weathering close to steep valley sides, the rock mass permeability can be high.

As stated at the start of this chapter, fresh volcanic rocks are used widely as construction materials. However, columnar jointed rock can be difficult to quarry, especially when the columns are almost vertical. Blast holes drilled vertically tend to jam in the open joints and explosive gases vent into them. The columns tend to topple over rather than fragment, resulting in a poorly graded (one-sized) quarry-run product. If the column diameter is large (i.e. more than 1 metre) then the product may be too large for the crusher. Regardless of the column size the poorly graded quarry product does not make good rockfill because it is difficult to compact and large voids remain between the rock blocks after compaction. This rockfill often has relatively low modulus and because of its large voids, requires special attention to filter design.

3.2.2.2 *Undersea flows*

Lava which has been extruded from the sea bed is often in the form of distorted globular masses up to 3 m long known as pillows (Figure 3.8).



Figure 3.8 Pillow structure in basalt, Pembrokeshire, U.K.

The pillows may be welded together, separated by fine cracks, or separated by sedimentary detritus derived from the lava. The long axes of the pillows are generally roughly parallel to the boundaries of the "flow". Bell (1983a) notes that joints, vesicules, and phenocrysts in the pillows are in some cases arranged radially. The joint pattern and the irregular shapes of the cracks at the pillow boundaries produce an overall fracture pattern that might at first appear to be "random".

3.2.3 Alteration of volcanic rocks

Fresh volcanic rocks are composed of minute, strong, tough mineral crystals, generally arranged in an extremely dense, interlocking manner. This structure results in negligible porosity and great strength and durability within the lifetime of engineering structures. However, as stated earlier, these rocks are often found to have been altered, probably during the late stages of solidification. In the altered rocks some of the crystals are wholly or partly changed to secondary minerals, including serpentine, calcite, chlorite, zeolites and clay minerals. These minerals are weaker and in some cases larger in volume than the original minerals, resulting in microcracking, increase in porosity and weakening of the rock. The altered rock is usually greenish in colour.

Where these effects are pronounced, and particularly if the clay mineral montmorillonite is produced, the altered rock is obviously much weaker than the original rock and is likely to deteriorate or even disintegrate on exposure to air or immersion in water. However if the alteration effects are relatively minor, e.g. only the margins of the original crystals are changed to secondary minerals, then the visual appearance of the rock substance may be little changed from that of fresh, unaltered rock. The altered rock will usually appear a little dull and have a slightly higher porosity and absorption than the fresh rock. Rocks such as this, showing only very minor and subtle alteration

effects, often deteriorate rapidly in pavements, are likely to be unsuitable for crushing to produce filters and may prove to be unsuitable for concrete aggregate and rockfill. Because of this, before adopting volcanic rocks for use as construction materials, they should always be subjected to very thorough checking, by local past performance and field observations, petrographic analysis and laboratory tests. An example is the calcite within the basalt filter aggregate at Laanecoorie Dam. For further details readers are referred to Shayan and Van Atta (1986), Van Atta and Ludowise (1976), Cole and Beresford (1976), Cole and Sandy (1980) and Hosking and Tubey (1969).

As the alteration is caused by hot water and gases which move through permeable features such as joints and highly vesicular zones, the more altered rock is usually located along and adjacent to such features. Quite commonly, secondary minerals occur as veins, seams or irregular masses filling previously gaping joints or voids. Such features, particularly where of large extent and composed of clay, serpentine or chlorite, represent significant rock mass defects with low shear strength.

3.2.4 Weathering of volcanic rocks

Although all unaltered volcanic rocks are highly durable within the life-span of normal engineering structures, the more basic varieties, particularly basalt, are quite susceptible to chemical weathering in a geological time frame. The distribution of weathered materials in volcanic rocks is governed by the distribution of any previously altered material, as well as by the pattern of joints and vesicular zones.

When extremely weathered, all volcanic rocks are clayey soils in the engineering sense. The acidic types tend to produce low or medium plasticity clays and the basic types high plasticity clays. Basalts commonly display spheroidal weathering profiles, with spheroids of fresh to distinctly weathered basalt surrounded by extremely weathered basalt, which is clay of high plasticity. Extremely weathered basalts and the surface residual soils developed on them are usually highly expansive and fissured.

3.2.5 Landsliding on slopes underlain by weathered basalt

The presence of a well developed pattern of slickensided fissures caused by shrink-swell from seasonal moisture changes may result in the shear strength of the mass being significantly lower than that of the intact material. Slopes steeper than about 10 degrees which are underlain by such materials often show geomorphological evidence of past or current landsliding.

Such landsliding occurs commonly at the steep margins of plateaus or hills capped by basalt flows which overlie old weathered land surfaces, as shown in Figure 3.9. The sliding occurs in some cases simply as a result of over-steepening of the hillside by erosion and in others due to pressure from groundwater exiting from a permeable zone beneath the extremely weathered basalt. The permeable zone may be jointed, less weathered basalt as in Figure 3.9 or alluvial sands or gravels on the old buried land surface. Examples are given in Fell (1992).

Landsliding in fissured, extremely weathered basalt in Victoria, Australia is described by MacGregor *et al.* (1990).

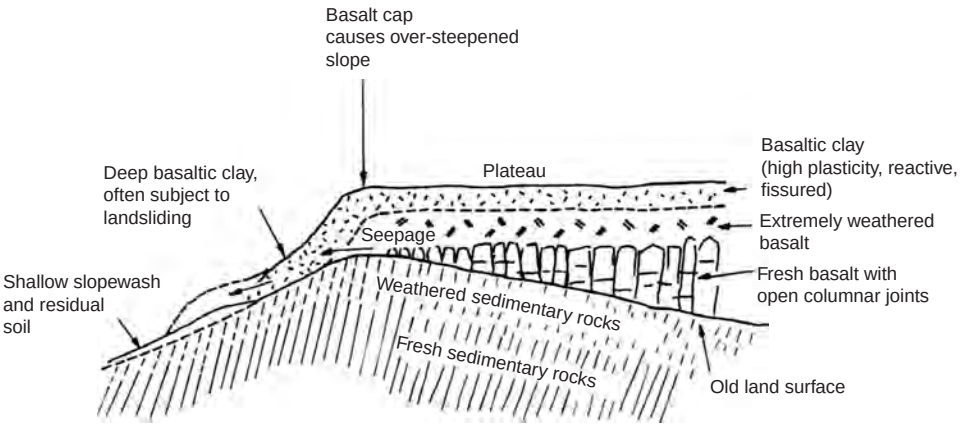


Figure 3.9 Typical profile through margin of basalt plateau, showing conditions which lead to slope instability.

3.2.6 Alkali-aggregate reaction

As discussed in Section 3.2 above, most volcanic rocks of Tertiary and younger ages contain appreciable amounts of glass, which may react with alkalis in Portland cement if the materials are used as aggregates in concrete. In older rocks, much or all of the glassy materials have usually developed a very fine crystalline structure and are less likely to be reactive. Zeolite minerals which occur in many volcanic rocks may also react with alkalis in cement.

Guillott (1975, 1986) and McConnell *et al.* (1950) describe observed effects of concrete expansion due to alkali aggregate reaction and discuss the mechanisms involved. Stark and De Puy (1987) describe observations and tests on affected concrete at five dams in USA. Cole and Horswill (1988) describe the deleterious effects and remedial work carried out at Val de la Mare dam, in Jersey, Channel Islands.

Shayan (1987) and Carse and Dux (1988) discuss some limitations of chemical and mortar bar tests for the prediction of the actual performance of aggregates in concrete.

The deleterious effects can be largely avoided by the use of low-alkali cement or pozzolanic additives in the concrete.

If volcanic rock is to be used as aggregate careful checking for possible alkali-aggregate activity is advisable, particularly if low-alkali cement is not available. Studies usually include checking of past performance, petrographic examination (ASTM, 1974a), and laboratory testing including the Quick Chemical Test (ASTM, 1974b or Standards Association of Australia, 1974a), the Gel-Pat Test (Building Research Station, 1958) and the Mortar Bar Test (ASTM, 1974c or Standards Association of Australia, 1974b).

3.2.7 Volcanic rocks (intrusive and flow) check list of questions

– Vesicular zones?
– "Clinker" or "breccia" zones?

- Lava tunnels?
- Old weathered/soil profiles?
- High mass permeability?
- Interbedded pyroclastic or sedimentary materials?
- Columnar joint pattern?
- Toppling failure?
- Difficulties in blasthole drilling?
- Poor fragmentation during blasting?
- Irregular joint pattern and "pillow" structure?
- Alteration effects – secondary minerals?
- Fresh, extremely strong boulders in extremely weathered materials (high plasticity clay)?
- Very high plasticity soils, expansive, fissured?
- Unstable slopes?
- Alkali-aggregate reaction?

3.3 PYROCLASTICS

Pyroclastic or "fire-broken" deposits are those which have been formed by the accumulation of solid fragments of volcanic rock, shot into the air during volcanic eruptions. The rock fragments include dense, solidified lava, highly vesicular lava (termed scoria) and extremely vesicular lava (termed pumice). Pumice is formed only from acidic lavas (e.g. rhyolite) and is so porous that it will float on water. Francis (1976) provides a detailed account of the ways in which pyroclastic materials are formed, based mainly on historical accounts of modern eruptions. Prebble (1983) describes the pyroclastic deposits of the Taupo Volcanic Zone in New Zealand and the difficulties they present in dam and canal engineering.

3.3.1 Variability of pyroclastic materials and masses

Pyroclastic deposits are characterized by extreme variability in engineering properties over short distances laterally and vertically. They range from extremely low density "collapsing" type soils to extremely strong rocks. This wide range in properties results from differences between the ways in which they were initially deposited and also from the ways they have been modified since deposition.

There are four main types of deposit, based on initial mode of deposition:

(a) Air fall deposits in which the fragments have simply been shot up into the air and fallen down again. Where they have "soil" properties such deposits are termed ash (sand sizes and smaller), or lapilli and bombs (gravel sizes and larger). Where welded, compacted or cemented to form rocks, they are termed tuff (sand sizes and smaller) or agglomerate (gravel sizes and larger in a matrix of ash or tuff).

(b) *Water-sorted deposits* in which the fragments fall into the sea or a lake and become intermixed and interbedded with marine or lake deposits. These also may be "soils" or rocks depending upon their subsequent history.

(c) *Air-flow, or "nuées ardentes", deposits* in which the fragments are white-hot and mixed with large volumes of hot gases, to form fluidized mixtures which can travel large distances across the countryside at speeds of probably several hundreds of kilometres per hour. The resulting materials, known as ignimbrites, range from extremely low density soils with void ratios as high as 5 (Prebble, 1983) to extremely strong rocks. The latter are formed when the white-hot fragments become welded together to form rocks almost indistinguishable from solidified lavas. These rocks are called welded ignimbrites or welded tuffs.

(d) Hot avalanche deposits which are formed by the gravitational breakup and collapse of molten lavas on steep slopes. The deposits comprise loosely packed but partly welded boulders, often showing prismatic fracture patterns which indicates that they were chilled rapidly after deposition (Francis, 1976).

Following or during their initial deposition, any of these types of deposit may be modified greatly by any or all of the following, which prevail in active volcanic environments:

- Faulting,
- Intrusion of further igneous material in plugs or dykes,
- Lava flows,
- Intense thunderstorm activity which causes erosion and redeposition, landsliding and mudflows,
- Hydrothermal alteration and chemical weathering.

The alteration and weathering produce clay minerals which may include montmorillonite, noted for its low shear strength, nontronite and allophane and halloysite which, in some cases, are highly sensitive.

The air-flow and air-fall deposits have commonly been deposited on old land surfaces with variable relief and covered by residual soil and weathered rock profiles. Prebble (1983) notes that such deposition in the Taupo Volcanic Zone of New Zealand resulted in permeable sand and gravel sized materials burying weak and relatively impermeable residual clays in old valleys. Subsequent cycles of deposition and near-surface weathering have resulted in a "valley-upon-valley" sequence of aquifers, interlayered with clayey aquicludes which tend to be sensitive and collapsible (see Figure 3.10).

Pyroclastic materials are also found in near-vertical pipes or necks, called diatremes. The Kimberley diamond pipe in South Africa and the Prospect Diatreme in Sydney, New South Wales (Herbert, 1983) are examples. The materials in the diatremes comprise angular fragments of volcanic rock plus fragments of the underlying and surrounding rocks. It seems likely that diatremes have been formed by explosive volcanic eruptions.

All of the above processes result in the extreme variability found in modern volcanic deposits. In geologically old deposits, which have been deeply buried by later sedimentation and folded, faulted and uplifted, new defects and variabilities are introduced, but the effects of compaction and consolidation can cause the strength contrasts between the various pyroclastic substances to be greatly reduced.

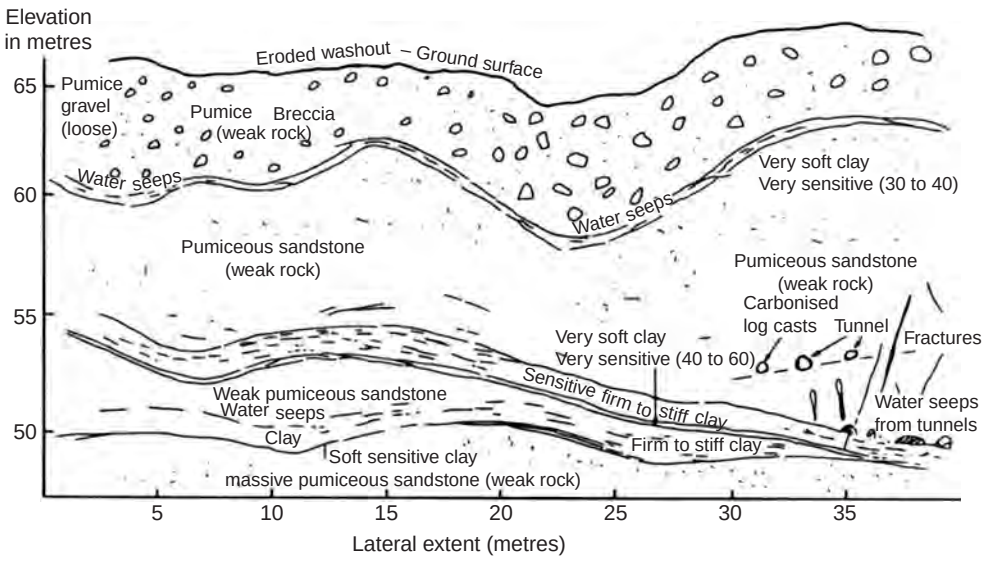
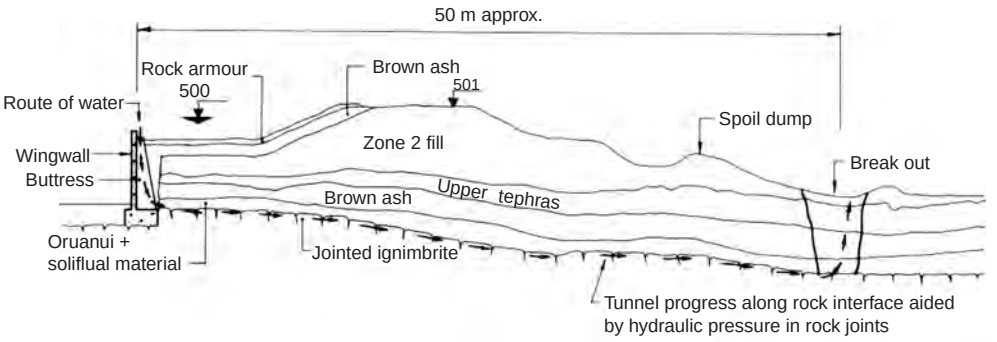


Figure 3.10 Exposure of pyroclastic materials in the collapsed area of Ruahihi Canal, New Zealand (Prebble, 1983).

3.3.2 Particular construction issues in pyroclastics

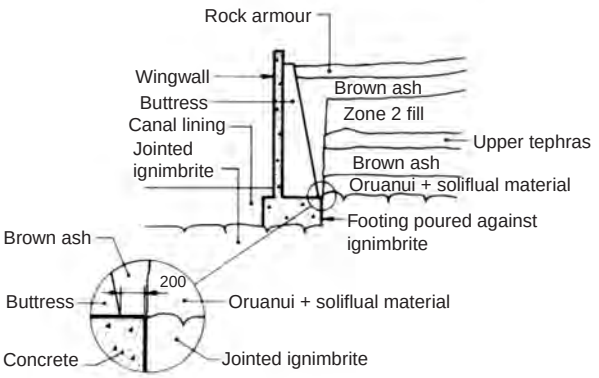
Prebble (1983), Jones (1988) and Oborn (1988) describe problems which were encountered at the Ruahihi and Wheao hydroelectric projects, constructed in the Taupo Volcanic Zone. Headrace canals for each project were constructed by cut and fill methods, on and through highly variable ash and ignimbrite deposits. Both canals failed by piping and subsequent collapse during early operation, apparently due to the high erodibility of some of the soils, both *in situ* and when compacted and to their brittle, non-healing nature which enabled the development of erosion tunnels. Oborn (1988) suggests that some of the soils at Ruahihi were probably dispersive and that accelerated rates of settlement after canal filling may have been due to the collapse of very low density soils on saturation after loading. At the Wheao failure area, erodible ash soils were located above very high strength welded ignimbrite with a columnar joint pattern. Near the upper surface of the ignimbrite, the joints were "bridged" by infill soils from above, but below this they were open as much as 50 mm. This feature was apparently missed during the construction stage cleanup. During operation, erodible ash soils were washed into these gaping joints, close to the penstock intake structure as shown on Figure 3.11, taken from Jones (1988). Prebble (1983) and Oborn (1988). Note also that the extreme sensitivity (up to about 60) of some of the alteration products (allophane and halloysite clays) caused problems during construction of the canals. Prebble predicts that these soils could collapse and liquefy when disturbed by earthquake loading or changed groundwater levels.

Jacquet (1990) describes the results of comprehensive laboratory tests on andesitic ash soils from seven sites in New Zealand. The soils contained high proportions of



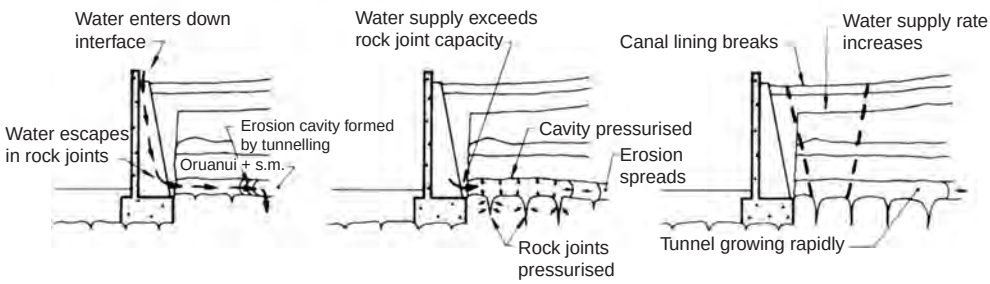
General arrangement looking upstream

Scale: 10 2 4 6 8 10 12 14 16 18 m



Detail of wingwall & buttress

Scale: 1 0 1 2 3 4 5 6 7 8 9 m



Phase 1. (early Dec. 1982)

Water exploits interface; penetrates remainder of lining and starts to erode Oruanui ash into rock joints; tunnels form.

Phase 2. (late Dec. 1982)

Water supply exceeds rock joint capacity; pressure in erosion cavity rises and assists spread of erosion.

Phase 3. (Shortly before 7.55 am 30.12.82)

Lining above tunnel breaks; water supply increases greatly; tunnels and rock exposed to full canal head; tunnel advances rapidly along favourable joints, assisted by water pressure.

Stages of development

Figure 3.11 Wheao Canal failure. Diagrams showing the general arrangement and how the failures developed (Jones, 1988).

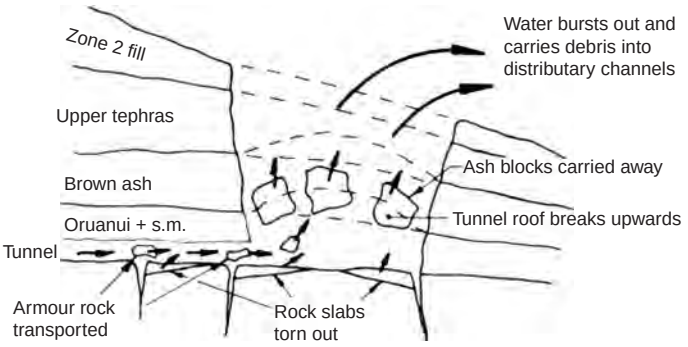

Zone 2 fill
Upper tephras
Brown ash
Oruanui + s.m.
Tunnel
Armour rock transported
Rock slabs torn out
Water bursts out and carries debris into distributary channels
Ash blocks carried away
Tunnel roof breaks upwards
Phase 4. (Break out – 7.55 am 30·12·82)
Tunnel reaches 50 m; roof bursts open; rock slabs torn loose and debris ejected into the distributary channels.
Stages of development

Figure 3.11 Continued.

allophane or halloysite and all classified as MH in the Unified System. Sensitivities ranged from 5 to 55. Jacquet concluded that the sensitivity was associated with irreversible rupture of the structural fabric of the soils and was not directly related to the clay mineralogy or classification characteristics of the soils.

Not all weathered pyroclastic materials are sensitive. In weathered agglomerates at the site of Sirinumu Dam in Papua New Guinea the matrix soils, which are clays of medium to high plasticity with 40 to 50 percent moisture content, are very resistant to erosion. The clay mineral types include halloysite, kaolin and allophane.

Rouse (1990) describes tropically weathered andesitic and dacitic ash soils from Dominica, West Indies, which occur on generally stable slopes of 30° to more than 50°. The soils are mainly allophane and halloysite clays, with very high residual friction angles (most between 25° and 35°).

Some unweathered non-welded ash and weakly-welded ignimbrite materials can be used as sand and gravel sized embankment filling, the weakly welded materials breaking down readily during compaction. However based on the experience at the 60 m high Matahina rockfill dam in New Zealand (Sherard, 1973) it is suggested that such materials should not be used for filter zones unless it can be shown that they will remain cohesionless in the long term. At Matahina weakly welded, partly weathered ignimbrite was compacted to form "transition" zones between the impervious core and rockfill zones. Subsequent excavation through the compacted ignimbrite showed it to have developed appreciable cohesion and it appeared to behave like a very low strength rock. This strength developed due to the interlocking of needle-shaped particles of glass.

This cohesive, brittle behaviour was an important contributing factor to the piping incident which occurred during the first filling of the reservoir in January 1967. The main cause of this incident, which resulted in the loss of more than 100 m^3 of core and

transition materials into the rockfill, was differential settlement over steps in slope correction concrete as shown in Figure 8.15 (Gillon, 2007).Other factors, as described by Sherard (1972, 1973) and Gillon and Newton (1988), included the possible reinforcement of the transition zones by means of cement grout, the erodible nature of the core material and large voids in the rockfill shoulders which were formed by poorly graded very strong welded ignimbrite, quarried from a columnar-jointed mass. Most blocks in the rockfill are of blocky shape and in the range 300 mm to 600 mm (Section 3.2.2.1).

As with lavas, pyroclastic materials, especially those of Tertiary or younger ages, contain glassy materials which may react with alkalis in Portland cement. They should therefore be tested thoroughly before use as aggregates in concrete (Section 3.2.6).

3.3.3 Pyroclastic materials – check list of questions

- Extreme variability?
- Very low *in situ* densities – collapse type behaviour?
- High *in situ* permeability?
- Brittle *in situ* and when compacted?
- Highly erodible *in situ* and when compacted?
- Highly to extremely sensitive zones?
- Complex groundwater distribution?
- Welded rocks: gaping joints?
- Columnar jointed welded rocks: poorly graded rockfill, quarrying problems?
- Interbedded lavas?
- Intrusive dykes, sills or plugs?
- Alkali-aggregate reaction?

3.4 SCHISTOSE ROCKS

Included in this group are those metamorphic rocks, e.g. slate, phyllite and schist which have developed a pronounced cleavage or planar foliation. The cleavage or foliation results from the parallel arrangement of platy minerals, commonly clays, muscovite, biotite, chlorite and sericite. Also often present and in parallel arrangement, are tabular or elongate clusters of other minerals, usually quartz and felspars and occasionally amphiboles.

Although the foliation is referred to as "planar", the foliae or layers are commonly folded. The folds can range in amplitude and wavelength from microscopic up to hundreds of metres. Small-scale folds, which cause the surfaces of hand-specimens of schist to appear rough or corrugated, are called crenulations.

In some schists the foliation has been so tightly and irregularly folded as to give a contorted appearance. Such rock is called "knotted schist".

Slate, phyllite and most mica schists have been formed by the regional metamorphism of fine-grained sedimentary rocks (mudstones or siltstones). This has involved relatively high temperatures and directed pressure over long periods of geological time. Under these conditions the clay minerals present in the original rocks have changed partly or wholly to mica minerals, usually muscovite, chlorite and biotite. These minerals have become aligned normal to the direction of the maximum compressive stress. The proportion of these new minerals is least in slate and greatest in schist.

Greenschists, which are well foliated rocks containing a large proportion of chlorite and other green minerals, have been formed by the regional metamorphism of basic igneous rocks (e.g. basalt and gabbro).

Schists can be formed also as a result of shear stresses applied over long periods of geological time to igneous rocks or to "high-grade" metamorphic rocks, e.g. gneisses. This process is known as retrograde metamorphism and usually produces schist containing abundant sericite and/or chlorite. These are weak minerals and so the rocks produced by this process tend to be weaker than other schists.

3.4.1 Properties of fresh schistose rock substances

Most schistose rocks have dry strengths in the very weak to medium strong range (see Table 2.4). Schists containing abundant quartz are generally medium strong or strong, while greenschists which are rich in chlorite are generally very weak.

The most significant engineering characteristic of schistose rocks is their pronounced anisotropy, caused by the cleavage or foliation. Figure 3.12 taken from Trudinger (1973) shows the results of unconfined compressive tests on fresh schist samples from Kangaroo Creek Dam, South Australia. This schist is stronger than most; it contains generally about 40% quartz and felspar and about 60% sericite and chlorite. It is foliated but not exceptionally fissile. It can be seen that the strengths recorded for samples loaded at about 45° to the foliation were about one third of those for samples loaded at right angles, i.e. the anisotropy index of this schist in unconfined compression is about 3.

When tested by the point load method, i.e. in induced tension, the schist at Kangaroo Creek shows anisotropy indices ranging typically from 5 to 10. Failure along the foliation surfaces in this test is by tensile splitting, rather than in shear.

Most other schists, slates and phyllites show similar anisotropic properties (Donath, 1961). It is clear that foliation angles in relation to loading directions should always be carefully recorded during tests and reported with the results.

In weak or very weak schists (often those rich in chlorite) the effective angle of friction along foliation surfaces can be low. Landsliding is prevalent in areas underlain by these rocks.

In knotted schists the foliation surfaces are often so contorted that shearing or splitting along near-planar surfaces is not possible. As a result, knotted schists are usually appreciably stronger than those in which the foliation surfaces are near-planar.

Figure 3.12 also indicates that the schist at Kangaroo Creek Dam showed a 25–65% reduction in strength, after soaking in water for 1–2 weeks. The greatest strength reduction occurred in the samples loaded at about 45° to the foliation. This result is typical of schistose rocks.

3.4.2 Weathered products and profiles developed in schistose rock

Schistose rocks vary widely in their susceptibility to chemical weathering. Varieties rich in quartz are very resistant and, at the other extreme, rocks rich in clay minerals or chlorite are very susceptible. Because of this, no typical weathered profile exists for all schistose rocks. However many schistose rock masses consist of layers (parallel to

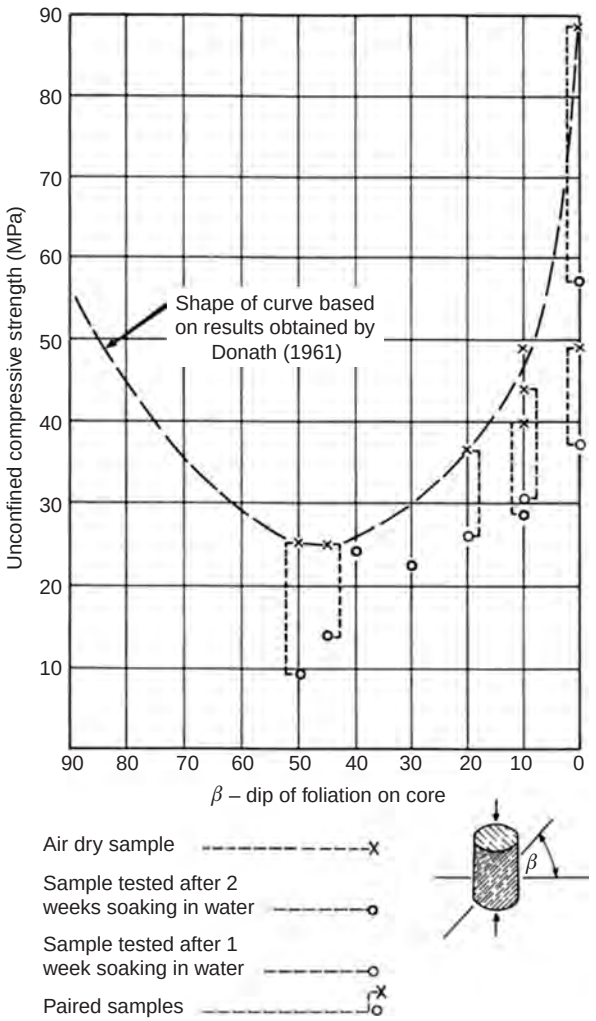


Figure 3.12 Variation of unconfined compressive strength of schist with angle between the foliation planes and applied loads (Trudinger, 1973).

the foliation) of varying susceptibilities to weathering. Thus weathering often tends to exaggerate the strength anisotrophy of such masses and the upper surface of fresh rock often has a deeply slotted or serrated shape, as shown in Figure 3.13.

Schists which are rich in micaceous minerals (biotite, muscovite or chlorite) tend to form micaceous silty or clayey soils when extremely weathered. The silty varieties are often of low *in situ* density and are highly erodible by water or wind. Also they tend to be hydrophobic, making dust control difficult on construction sites.

Even when fresh or only partly weathered, the more micaceous, fissile schists usually produce much dust due to abrasion during handling and trafficking.

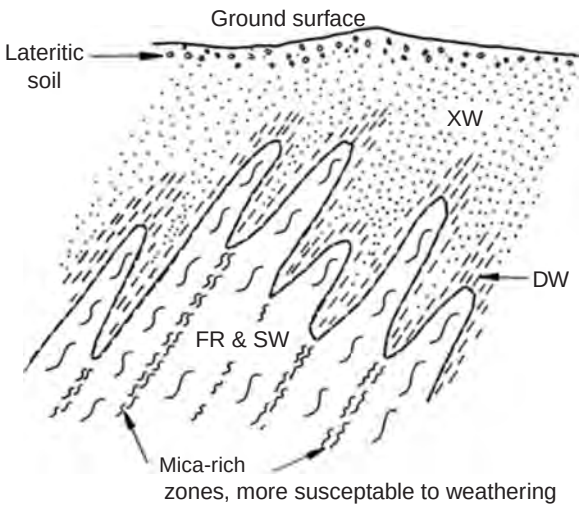


Figure 3.13 Weathered profile developed on schistose rock with steeply dipping foliation.

3.4.3 Suitability of schistose rocks for use as filter materials, concrete aggregates and pavement materials

Schistose rocks are generally unsuitable for any of these purposes due to the very flaky shapes of the crushed materials and inadequate strengths of the particles. Kammer and Carlson (1941) describe the unsuccessful use of phyllite as aggregate for concrete in a hydroelectric plant. Silicates in the phyllite reacted with alkalis in the cement to cause expansion and disruption of the concrete. The strongest, most siliceous schists have been used successfully in base courses of pavements, where more suitable materials have not been available.

3.4.4 Suitability of schistose rocks for use as rockfill

Despite their tendency to produce very platy block shapes, schistose rocks have been used successfully as rockfill on several dams up to 80 m high. At Kanmantoo Mine in South Australia a 28 m high rockfill dam with a thin sloping earth core was built in 1971 for the storage of tailings and water for use at the mine (Stapledon *et al*., 1978). The rockfill was mainly very weak quartz-biotite schist, the waste rock from the mine. It was placed in 0.6–0.9 m thick layers and compacted dry with a Caterpillar D8 tractor and by trafficking by the 50-tonne dump trucks. Post-construction crest settlements 5.5 years after completion ranged from 0.3% to 0.7% of the embankment height.



Figure 3.14 Schist being quarried in the spillway excavation at Kangaroo Creek Dam. Note the platy shapes of the rock fragments.

The successful use of schist as rockfill at Kangaroo Creek Dam has been described by Good (1976) and Trudinger (1973). Trudinger describes in some detail the construction procedures, the behaviour of the rock and the fill densities obtained. The concrete faced dam as built initially was 60 m high. Zone 3, which contained about 300,000 m^3 of the 443,000 m^3 total rock in the embankment, was built mainly of weak to medium strong, slightly weathered schist compacted in 1 m layers. The maximum specified block size was 1 m, but many blocks were longer than this due to the very platy shapes obtained during quarrying (Figure 3.14). Because the schist suffered a large strength loss on saturation (Figure 3.12) the rock was heavily watered during placement. During compaction by 4 passes of a 10-tonne vibrating roller, the uppermost 50–300 mm of most layers were crushed into gravelly sandy silt (Figure 3.15). These layers formed thin, lower permeability barriers but were readily penetrated by the basal rocks of the next layer and hence they did not form weak or compressible zones in the dam. However, the presence of these thin, low permeability layers, which occur within all zones of rockfill in Kangaroo Creek, has resulted in a significant drop in the vertical permeability of all rockfill zones, including the two zones specifically required to provide a pathway for water to the bottom drainage zones. If such rockfill is to be used in concrete face rockfill dams or where large drainage capacity is required it is important to include zones of more permeable rockfill from selected rock from the quarry or from another rock source.

After 20 years of operation the dam height was increased to 64 m by placing a rock-filled reinforced concrete trough across its crest.

In 2003, thirty four years after its initial completion, the maximum creep settlement at the original crest level was 184 mm, which is 0.3% of the initial height and 0.29% of the revised height.



Figure 3.15 Slurry (Gravelly sandy silt) formed on the top of schist rockfill layer as a result of weathering and compaction by 4 passes of a 10-tonne vibrating roller (Trudinger, 1973).

3.4.5 Structural defects of particular significance in schistose rocks

Three defect types are discussed below.

3.4.5.1 *Minor faults developed parallel and at acute angles to the foliation*

Schists commonly contain minor faults (narrow, sheared zones or crushed seams, or both) parallel to the foliation. Deere (1973) refers to these features as foliation shears. In folded schists, the foliation shears have probably been formed by inter-layer slip, as shown on Figure 3.16.

Also present in many folded schists are similar "shears" cutting across the foliation at acute angles, generally less than 20° (Figure 3.16). In some cases these are thrust faults. Residual shear strengths of both foliation shears and cross-cutting faults have been found in laboratory tests (i.e. excluding the effects of large scale roughness), to lie in the range 7° to 15°. Such defects commonly form the initiating failure surfaces of landslides in schistose rocks (Figure 3.17) and may provide potential sliding

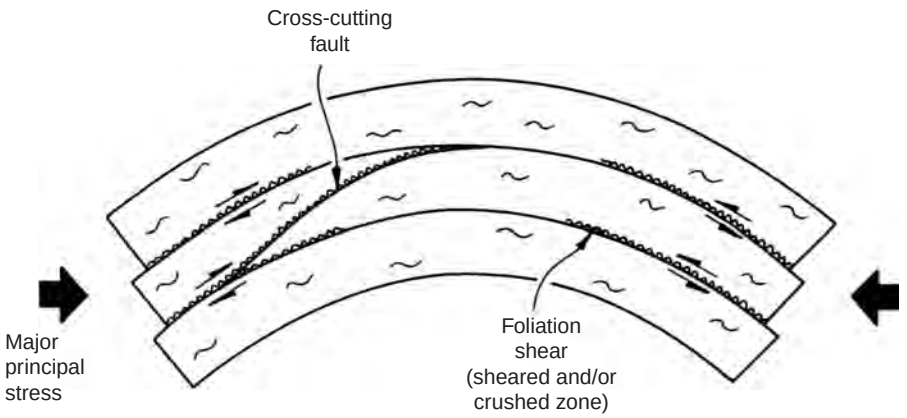


Figure 3.16 Probable way in which foliation shears and cross-cutting faults are formed in some schistose rocks.

surfaces into spillway or foundation excavations, or within the foundations of an embankment dam.

As these features are often only 50 mm or so in thickness, they can escape detection during site investigations unless the investigator sets out to look for them, using appropriate techniques, e.g. well cleaned up trenches and high quality core drilling. The sheared zones can be particularly difficult to detect. In these zones the rock is more intensely foliated than elsewhere and is usually rich in chlorite and/or sericite. The sheared material is therefore appreciably weaker than the normal schist and is readily recognizable when the rock is fresh. However, in distinctly or extremely weathered exposures in which both sheared and unsheared materials are greatly weakened by weathering, it can be quite difficult to recognize the sheared zones, because the strength contrast is much reduced, and the shear- induced cleavage or foliation is similar in appearance to, and may be parallel to, the foliation in the normal schist. Stapledon (1967) describes how initial, poor quality core drilling at Kangaroo Creek (South Australia) failed to indicate the presence of foliation shears and associated infill clay seams. Discovery of these in a later exploration programme led to abandonment of a thin concrete arch design in favour of a concrete face rockfill dam. Paterson *et al.* (1983) describe how surface mapping and diamond drilling were not adequate to define the full extent and frequency of foliation shears at the site for Clyde Dam in New Zealand. The location and treatment of some of these shears during construction of the dam are discussed in Section 17.8.3.

3.4.5.2 *Kink bands*

Schistose rocks often also contain "kink bands" within which the foliation layers have been displaced to form features similar to monoclinal folds but with sharp, angular hinges (Figure 3.18a).

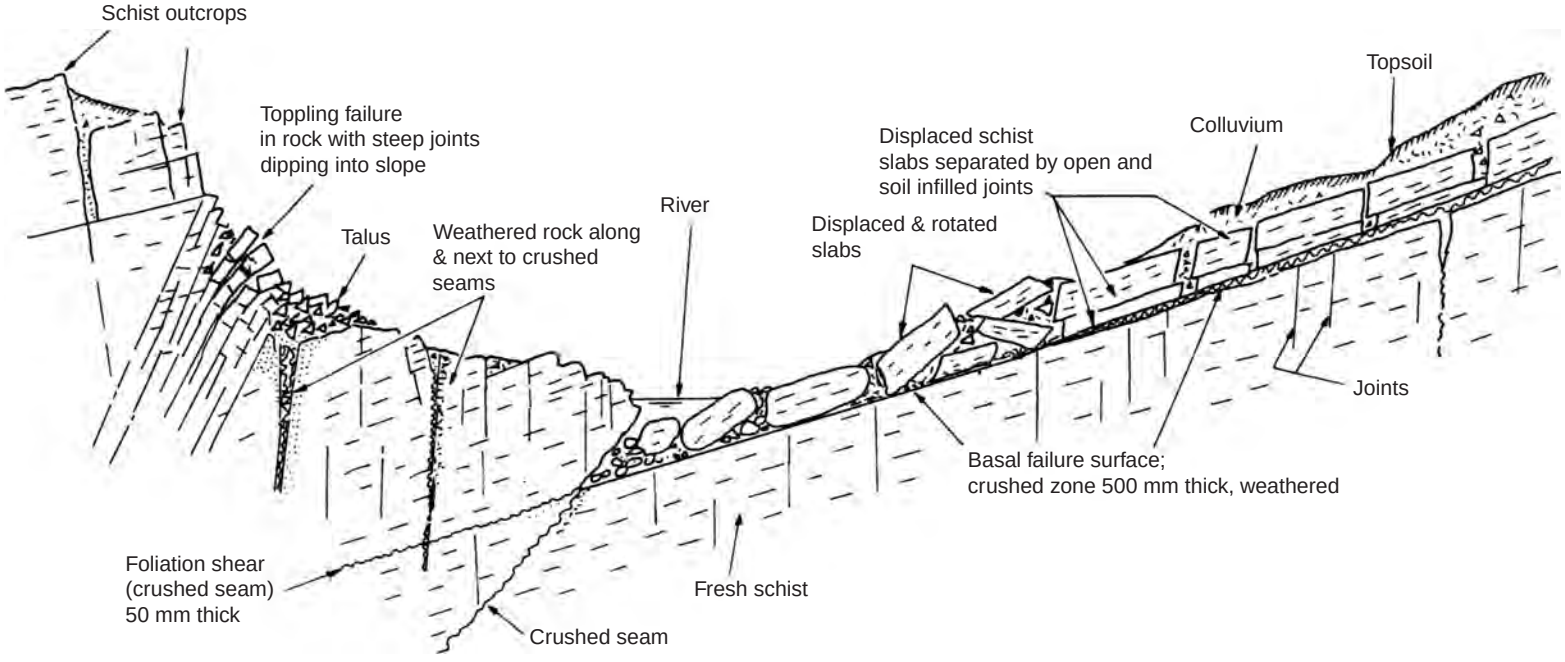


Figure 3.17 Typical valley profile in gently dipping schist affected by past landsliding along foliation shear.

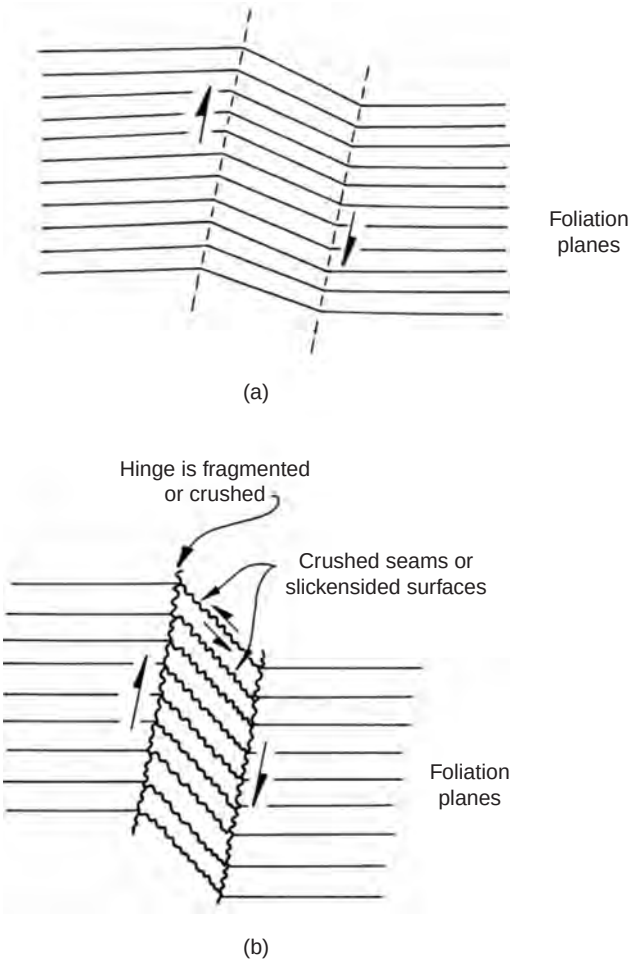


Figure 3.18 Formation and nature of kink bands.

In some cases continuous near-planar joints have developed along the hinges and the foliation layers within the band have parted to form open or infilled joints. In other cases crushed seams have developed along the hinges and the foliation layers within the band are either slickensided or partly crushed (Figure 3.18b). Such kink bands can be considered a special class of fault.

3.4.5.3 *Mica-rich layers*

Some schists contain layers or zones which consist almost entirely of micaceous minerals, e.g. biotite, muscovite, sericite or chlorite. Such layers or zones are usually much weaker than the normal schist. It is good practice to consider them as individual defects of low shear strength.

3.4.6 Stability of slopes formed by schistose rocks

Landsliding is relatively common on slopes underlain by schistose rocks. Well-known examples are the Madison Canyon Rockslide in Montana, U.S.A. (Hadley, 1978), the Downie Slide in British Columbia (Piteau *et al*., 1978) and the Tablachaca Slide in Peru (Arnao *et al*., 1984; Deere and Perez, 1985).

The landsliding usually occurs by slope failure along weathered foliation surfaces or foliation shears as shown on Figure 3.17. Bell (1976, 1982) describes how the development of a river valley in New Zealand has involved landsliding along dipping foliation surfaces.

In steeply dipping schistose rocks failure by toppling is also relatively common, the toppled slabs or columns being separated by joints or shears along the foliation direction. Riemer *et al*. (1988) describe a complex toppling failure at San Pablo, Peru.

Where the schistose rocks contain abundant tectonically-formed defects in other orientations, many other failure models have been recorded. However in most of these it is likely that the low shear strength of the schistose rocks along their foliation surfaces has contributed to the development of slope failure.

In long, high slopes in mountainous areas some failures of schistose rocks appear to have occurred by buckling of, and eventually shearing through, the foliation. The buckling is facilitated by the low shear strength of the foliation surfaces which allows multiple shear displacements to occur along them. Examples of this type of slope failure are discussed in Beetham *et al*. (1991), Riemer *et al*. (1988), Radbruch-Hall *et al*. (1976), Nemcok (1972) and Zischinski (1966, 1969).

Examples of landsliding in schists forming slopes around the reservoir of Clyde Dam in New Zealand are discussed in Gillon and Hancox (1992), Riddolls *et al*. (1992) and Stapledon (1995).

3.4.7 Schistose rocks – check list of questions

- Degree of anisotropy, and its effect on the project?
- Low durability in exposed faces?
- Particle shapes and strengths inadequate for filter, concrete or pavement materials?
- Suitability for use as rockfill?
- Foliation shears?
- Kink bands?
- Mica-rich layers?
- Unstable slopes?

3.5 MUDROCKS

Included under this heading are all sedimentary rocks formed by the consolidation and cementation of sediments which are predominantly clays or silts or clay-silt admixtures. The common rock types are:

- Claystone (predominantly clay sizes),
- Siltstone (predominantly silt sizes),

– Mudstone (clay-silt admixtures),
– Shale (any of the above, but fissile due to well-developed cleavage parallel to the bedding).

The sediments may have been deposited in either marine or fresh water conditions and usually have been derived from erosion of older rocks. In some cases they contain particles of volcanic origin. Possible cementing agents include calcite, silica, iron oxides and evaporite minerals such as gypsum, anhydrite and halite (common salt). Mudrocks occurring in or associated with coal-bearing sequences often contain abundant carbonaceous material and sulphide minerals, e.g. iron pyrite.

3.5.1 Engineering properties of mudrocks

Most mudrocks when fresh lie in the weak to very weak range as defined on Table 2.4. The very weak claystones grade into hard, overconsolidated clays. The strongest mudrocks lie in the medium strong and strong ranges and in most cases these owe their greater strength to cementation by calcite or silica.

Because of their relatively high clay contents the porosity and water absorption properties of mudrocks are much higher than those of most other rocks. As a result of this and the expansive nature of clays, all mudrocks swell and develop fine cracks on prolonged exposure to wetting and drying. The strongest siltstones, which contain appreciable amounts of calcite or silica as cement, can be exposed for up to a year before cracks are evident. At the other end of the scale, the weakest claystones and shales develop fine cracks as soon as they dry out (often only hours or days) and disintegrate with further cycles of wetting and drying. Some of these materials also swell noticeably on removal of overburden.

The mechanisms involved in deterioration of mudrocks on exposure have been described already in Sections 2.5.3 and 2.9.1.

Because of their instability when exposed, special care needs to be taken during preparation of foundations on mudrocks. Treatments range from shotcreting or slush concreting immediately after exposure and cleanup, to an initial cleanup followed by a final cleanup immediately before placement of concrete or fill.

Taylor and Spears (1981), Cripps and Taylor (1981) and Spink and Norbury (1993) provide useful information about the engineering properties of mudrocks occurring in the United Kingdom.

Mudrocks containing iron pyrite or other sulphide minerals can cause severe problems for dam projects due to rapid weathering of the sulphides to form sulphuric acid and metallic hydroxides and sulphates. The rapid weathering processes and some of their effects on dam projects are described in Section 2.9.4. Such effects may include:

– Damage to the fabric of concrete due to sulphate attack,
– Heave of foundations or of excavation sides causing damage to concrete slabs or walls (Penner *et al*., 1973; Hawkins and Pinches, 1987),
– Blockage and/or cementation of filter zones or drains,
– Seepage waters which are acidic and rich in iron or other heavy metals,
– Possible lowering of shear strength, of embankments formed by pyritic mudrocks.

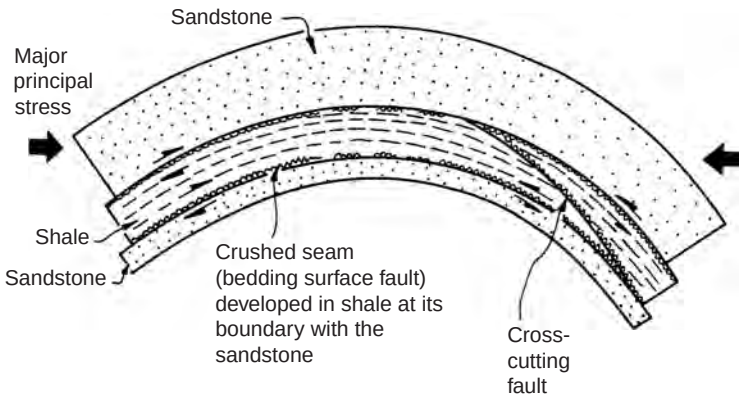


Figure 3.19 Usual way in which bedding surface faults are formed in mudrocks.

3.5.2 Bedding-surface faults in mudrocks

Valley bulging (see Chapter 2, Section 2.5.4 and Figure 2.7) is a common feature in mudrock sequences, where the beds are near horizontal. In such situations and in any other situation in which mudrock sequences have been disturbed by folding, tilting, or stress relief movements, thin seams of crushed rock develop within the mudrocks, usually at their boundaries with interbedded stiffer rocks (e.g. sandstones or limestones). These seams are developed due to interbed slip during the movements, in the same way as foliation shears develop in schistose rocks (Figures 2.8, 3.16 and 3.19). In mudrocks they are known as bedding-surface faults, bedding-surface shears or bedding-plane shears. They usually consist mainly of clay, are almost planar and have slickensided surfaces both within them and at their boundaries. Residual effective shear strengths of laboratory sized samples (i.e. excluding larger scale roughness effects) are commonly in the range 7° to 12°, with zero cohesion.

Although usually extending over wide areas, bedding surface faults may be only a few millimetres thick. Such defects are difficult to recover and recognize in diamond drill cores. This is particularly so when the defect is normal to the axis of the borehole, as even minor core rotation causes remoulding of the defect, which is then difficult to distinguish from remoulded mudrock at a "drilling break" in previously intact core (see Section 3.5.6).

The influence of bedding surface faults on the design of several embankment dams is described by Casinader (1982). Their effects on the design and construction of Sugarloaf and Thomson Dams are discussed in Chapter 2, Section 2.10.

Maddox *et al*. (1967) describe their effects at Meadowbank Dam, a 43 m high massive buttress dam in Tasmania, founded on interbedded sandstones and mudstones. The dam was planned originally to be a concrete gravity structure but, during excavation of the foundation, continuous bedding-surface seams of clayey silt were found within the mudstones. Change to the buttress design and installation of prestressed cables were needed to achieve the required factor of safety against sliding.

3.5.3 Slickensided joints or fissures

Many mudrocks, particularly claystones, contain zones in which the rock contains an irregular network of curved, intersecting, slickensided joints. These joints are believed to have originated when the material was still a clay soil and to have developed by any of the following types of process:

- syneresis (Skempton and Northey, 1952; White, 1961),
- shrink and swell movements (Corte and Higashi, 1964),
- differential shear movements during consolidation,
- large lateral stresses (Aitchison, 1953; Terzaghi, 1961).

Due to their lack of continuity and their curved, irregular nature, these joints usually do not form continuous zones of very low shear strength. However, the shear strength of the jointed mass is appreciably lower than that of the intact mudrock. In adopting strength parameters for use in design, the strength of both the intact substance and the joints, and the spacing, orientation and continuity of the joints, need to be taken into account.

3.5.4 Weathered products and profiles in mudrocks

Weathering of mudrocks usually involves mechanical disintegration as described in Section 3.5.1 and the removal of cements such as calcite and silica. In the extremely weathered condition all mudrocks are clays or silts. Intermediate weathered conditions (e.g. slightly and distinctly) are often difficult to define in the weaker mudrocks, which when fresh are only a little stronger and more durable than hard clays. The Geological Society Engineering Group (1995) provides a suggested approach for site-specific mass-type classification of weathered profiles in these weaker mudrocks.

Weathered profiles in mudrocks are more uniform, gradational and generally not as deep as those in other rocks. Deere and Patton (1971) describe these types of weathered profiles in shale and point out that the lack of distinct boundaries within such profiles has led to contractual disputes over the depth to acceptable foundations.

3.5.5 Stability of slopes underlain by mudrocks

Slopes underlain by mudrocks commonly show evidence of past instability, even when the slope angles are small (e.g. 10° to 15°). This is not really surprising when we consider that all of the characteristics described above in Sections 3.5.1 to 3.5.4 tend to lower the strength of rock masses containing mudrocks.

Relatively shallow landsliding is common in the residual clay soils developed on the weaker mudrocks. Taylor and Cripps (1987) provide a comprehensive review of slope development and stability in weathered mudrocks and overconsolidated clays, mainly relating to examples in the United Kingdom.

Deere and Patton (1971) give a useful review of experience with unstable slopes on shales and on shales with interbedded sandstones. They point out that, in common with most other rocks, weathering of shales usually produces a low-permeability zone near the surface, underlain by jointed, less weathered shale, which is more permeable. Instability can arise when groundwater transmitted through this lower zone or along

sandstone beds causes excessive pore pressures in the near-surface, more weathered shale (See Section 3.6.4 and Figures 3.23 and 3.24).

The most common situations in which larger scale landsliding occurs, or is likely to occur, are where the bedding "daylights" on a valley slope. This occurred at both Sugarloaf and Thomson dams in Victoria, Australia, as described in Chapter 2, Section 2.10.

3.5.6 Development of unusually high pore pressures

Stroman *et al.* (1984) and Beene (1967) describe a major slide which occurred during the construction of a 30 m high section of the 5.5 km long Waco Dam in Texas. The dam was located on near-horizontally bedded shales cut through by three steeply dipping normal faults, which displaced the shale units by up to 30 m and caused them to be locally folded (Figure 3.20a).

The originally designed embankment is shown in outline on Figure 3.20(b). The design was based on the assumption that the weakest foundation material was a localized 12 m thick layer with unconsolidated undrained strength of $\phi = 5°$, $c = 144$ kPa. No potential for pore pressure development was expected (because of the high degree of overconsolidation of the shales) and consequently no piezometers were installed in the original construction.

The failure occurred in a 290 m long section of embankment constructed between two of the normal faults. As can be seen on Figure 3.20c, the failure surface was mainly horizontal within the Pepper Shale about 15 m below the main foundation level. This failure surface broke out to the ground surface at an average distance of 235 m downstream from the dam axis.

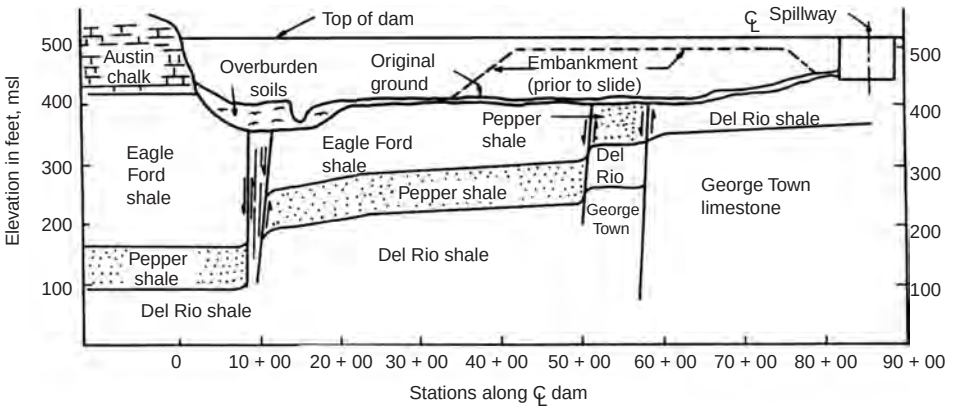
Figure 3.21 shows the pore pressure distributions before the sliding and at the end of the reconstruction. Stroman *et al.* (1984) state that the unusually high pore pressures at the Pepper Shale/del Rio Shale contact were the cause of the slide. However they also report that direct shear tests on precut samples of Pepper Shale gave effective friction angles between 7° and 9°, with zero cohesion. They state "*This is the laboratory test condition that can be related to a material that has been broken prior to construction and to the condition of the Pepper Shale after the slide*".

Londe (1982) considers that the sliding at Waco Dam occurred by progressive shear failure (e.g. as described by Bjerrum, 1967, Terzaghi and Peck, 1967, Skempton and Hutchinson, 1969 and subsequently by Skempton and Coates, 1985, to explain the failure of Carsington Dam).

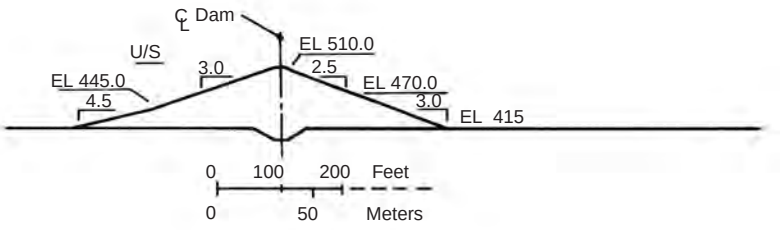
It seems equally possible to the present authors that very thin but continuous bedding surface faults with 7° to 9° residual strength may have existed in the Pepper Shale before the slide and contributed to the movement. Such bedding surface features require very small movements for their development and these could easily have occurred during the formation of the normal faults which bound the slide area. As described in Section 3.5.2, such minor bedding faults can be difficult to recover and recognize in drill cores.

3.5.7 Suitability of mudrocks for use as construction materials

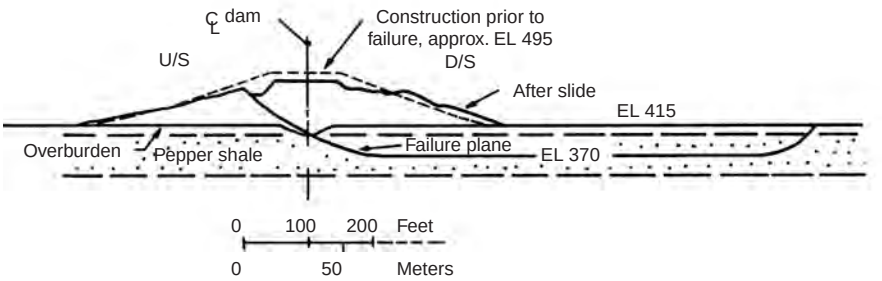
Most mudrocks are not suitable for the production of materials for use in concrete, filters or pavements due to their generally low strengths and their slaking properties.



(a) Longitudinal section (exaggerated vertical scale)



(b) Cross section original design



(c) Cross section showing failure surface

Figure 3.20 Longitudinal and cross sections, Waco Dam (Stroman *et al.*, 1984).

However, near Sydney, a trial road pavement built using ripped and trafficked shale has performed satisfactorily for 20 years (Won, 1985). The shale is weak to medium strong. Re-sampling and testing of the material showed that there had been little breakdown in service.

Siltstones in the medium strong to very strong range have been used successfully as rockfill for some years. Near the site of Sugarloaf Dam excavation of test pits into 100 year old tunnel spoil dumps comprising such materials, showed that slaking had

not occurred in the rock below the near-surface two metres, i.e. in a near constant humidity environment. ICOLD (2008) also found no evidence to support concerns that shale breaks down to form clay after placing in shoulder fills.

To the knowledge of the authors no mudrocks have been used successfully as riprap. We would not expect it.

Random fills, earthfills and cores for embankment dams have been built successfully using mudrocks in various conditions ranging from fresh to extremely weathered (Vaughan, 1994).

ICOLD (2008) recommend ripping the rock heavily before excavation from the borrow area, and heavy compaction of mudrocks if they are to be used as earthfill. They warn of potential collapse settlement of the upstream shoulder fill on saturation by the reservoir.

3.5.8 Mudrocks – check list of questions

- Slaking or disintegration on exposure?
- Swelling on exposure?
- Valley bulging?

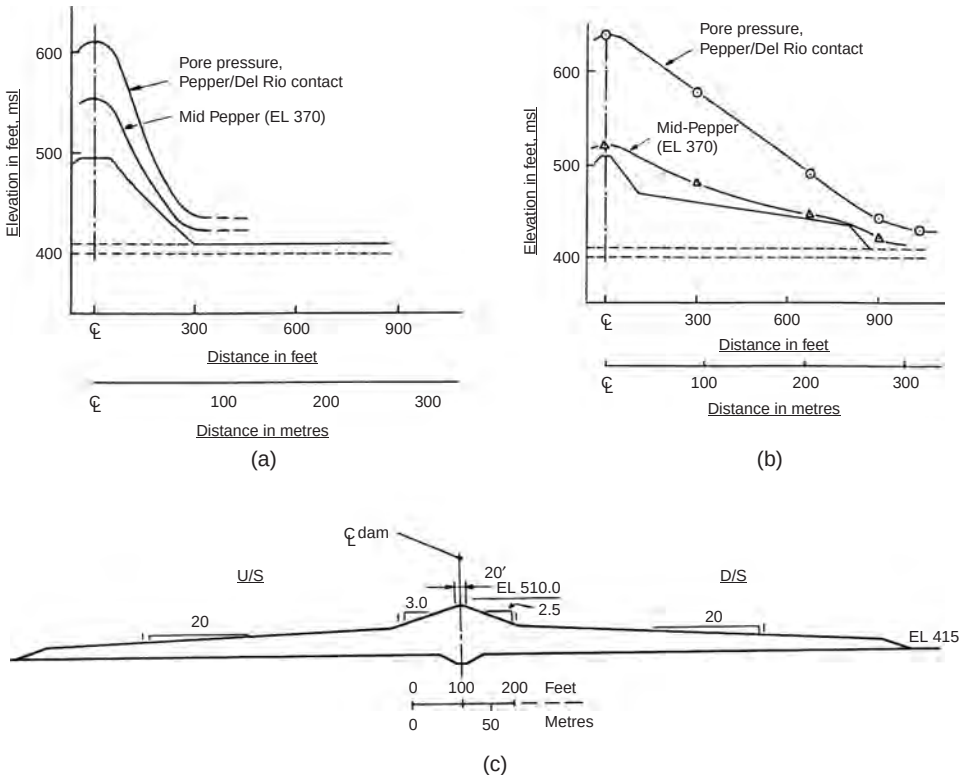


Figure 3.21 Waco Dam, pore pressure plots and cross section through embankment as rebuilt. (a) pore pressures prior to slide; (b) pore pressures after reconstruction; (c) embankment as reconstructed (Stroman *et al.*, 1984).

- Soluble minerals in beds or veins?
- Presence of sulphide minerals?
- Slickensided fissures?
- Progressive shear failure?
- Bedding surface faults or shears?
- Unstable slopes (shallow, in weathered materials)?
- Unstable slopes (deep-seated, if bedding in folded rocks daylights)?
- Possibility of high pore pressures, in layered sequences?
- Suitability for rockfill, random fill, earthfill and haul roads?

3.6 SANDSTONES AND RELATED SEDIMENTARY ROCKS

The following are the main rock types considered under this heading. They will be referred to as the sandstone group.

- Sandstones,
- Arkoses,
- Greywackes,
- Siltstones,
- Conglomerates.

Table 3.1 sets out some common characteristics of sandstones, arkoses and greywackes. Siltstones and conglomerates have a similar range in composition to those of the other rocks. All except the siltstones occur usually in thick beds. The siltstones may be thickly or thinly bedded.

Also included under this heading are the lightly metamorphosed equivalents of the above, e.g. quartzites, metasiltstones and metaconglomerates. If the original depositional environment of the particular rocks at a site is known, then many generalisations can be made about their mineral contents, fabrics, bed-thicknesses and sedimentary structures and about rock types likely to be associated with them. Detailed discussion of these sedimentological aspects is beyond the scope of this book and only a few features of particular importance in dam engineering will be described. For more details readers are referred to Pettijohn (1957), Pettijohn *et al.* (1972), Selley (1982) and Walker (1984).

Table 3.1 Common characteristics of sandstones, arkoses and greywackes.

		Minerals	
Rock name	*Particle shapes, grading*	*Most grains*	*Common matrix/cements*
Sandstone	Usually rounded, one-size grains and less than 15% matrix or cement	Quartz, fragments of older rocks	Silica, clay, iron oxides, calcite, gypsum
Arkose	Sub-angular, often well graded, little matrix	Quartz plus at least 25% feldspar; some mica	Clay, iron oxides, silica
Greywacke	Angular, well graded down to clay matrix which is usually >15% of volume	Felspar, quartz hornblende, micas, rock fragments, iron oxides	Clay, and same as grains

3.6.1 Properties of the rock substances

When fresh the sedimentary rocks range from extremely weak, non-durable to very strong and durable. The strengths and durabilities depend upon the strengths and durabilities of the grains and of the cements or matrices and these can vary widely depending upon the environment of deposition and subsequent histories of the rocks.

Quartz-sandstones (and conglomerates in which most grains are quartz) are often stronger and more durable than arkoses and their conglomerate equivalents, because of the superior strength and durability of quartz. However sandstones often have significant porosity (5% to 20%) and may also be slightly permeable.

Greywackes tend to be stronger than sandstones due to the angularity and grading of their particles.

Silica cement usually occurs in strong, durable rocks and at the other extreme rocks cemented by clay or gypsum are usually weak and non-durable.

If gypsum or anhydrite is proven or suspected as a cement in a sandstone forming all or part of the foundation of a dam, its significance needs to be assessed carefully and special testing may be required, as discussed in Section 3.8.

The metamorphic rocks when fresh are usually stronger and more dense and durable than the equivalent sedimentary rocks.

3.6.2 Suitability for use as construction materials

Rocks in the sandstone group which lie in the strong to extremely strong range have been used successfully as rockfill and rip-rap in many dams. They are widely used also as aggregates in concrete.

However, in a few cases concrete containing quartzite or strong sandstone as aggregate has suffered expansion and cracking due to alkali-silica reaction. The authors recommend that any silica rich rocks intended for use as concrete aggregate be tested for reactivity (see Section 3.2.6).

The quartzites are often extremely strong and this together with the high content of quartz (Moh hardness = 7) makes them highly abrasive. This can result in high quarrying and handling costs. Also if the quarry-run rock is not well graded it can be difficult to compact, as little breakdown occurs under the roller.

The weaker rocks tend to be more porous and usually lose significant strength on saturation. Mackenzie and McDonald (1981, 1985) describe the use of sandstones and siltstones, mainly medium strong when dry, as rockfill in the 80 m high Mangrove Creek Dam in New South Wales. Both rock types lost about 50% of their strength on saturation.

The rock was compacted with up to 5% water by volume. Higher water quantities caused the material to become unworkable. The fills produced were of high density and moduli but of generally low permeability. The latter was allowed for by the inclusion of drainage zones of basalt and high quality siltstone.

At Sugarloaf (Winneke) Dam thinly interbedded siltstone and sandstone were used for rockfill and random fill in the 85 m high concrete faced embankment (MMBW, 1981; Regan, 1980).

The rockfill zone material was fresh or slightly weathered and strong to medium strong. Compacted with up to 15% water it produced a dense, free-draining fill.

The rock used in the random fill zone was slightly to highly (or distinctly) weathered and medium strong to weak. Compacted at about 10% moisture content it

produced a dense fill which was not free-draining and contained up to 20% of silt and clay fines which were dispersive. It was therefore underlain everywhere by a blanket of high permeability rockfill and its outer surfaces were protected by a thin layer of the rockfill.

3.6.3 Weathering products

The main effect of chemical weathering on rocks of this group is weakening due to removal or decomposition of the cement or matrix. As a result the rocks with silica or iron oxide cements are the most resistant to weathering. The sandstones and conglomerates become progressively weaker until at the extremely weathered stage they are usually sands or gravels. Quartzites and quartz rich sandstones produce relatively clean quartz sands, while arkoses and greywackes usually produce clayey or silty sands. Siltstones usually produce clayey silts or clays.

Some sandstones (usually weak, porous types) are locally strengthened in the weathered zone, by the deposition of limonite in their pores.

It can be difficult in some sandstones (e.g. those with calcite or limonite cements) to distinguish effects of weathering from effects of processes involved in the formation of the unweathered rocks. This can make it impossible to classify the rocks in the usual weathered condition terms, e.g. those in Table 2.3.

3.6.4 Weathered profile and stability of slopes

Weathered profiles in the weaker, more porous rocks commonly show gradational boundaries between rock in various weathered conditions. This is also the case in the stronger, more durable rocks when they are closely jointed.

Where large contrasts occur between the resistance to weathering of interbedded rocks, sharp but irregular sawtooth shaped boundaries can occur as shown in Figure 2.23.

Figure 3.22a shows the type of mechanically weathered profile often developed close to cliffs formed bystress relief as the valley is formed in near-horizontal sequences of thick sandstone beds underlain by or interbedded with siltstones or shales. Crushed seams (bedding surface faults) occur along the bed boundaries and steeply dipping joints have opened up in the sandstones. If these effects only occur close to the side of the valley it is likely that they result mainly from inter-bed movements due to stress relief – the shales/siltstones expanding further out of the slope than the sandstones. Joint-water pressures during extreme rainfall periods and earthquake forces may also contribute to the slight movements of the sandstone blocks.

Opening up of the joints in the sandstones causes the permeability of layers near the surface to be greatly increased. Figure 3.22b shows an example of in-filled stress relief joints at Shannon Creek Dam.

Where whole hillsides are underlain entirely by rocks of the sandstone group, these rocks commonly form steep slopes or cliffs. Slope failures are rare and usually occur by rockfalls or toppling from the cliff portions.

More commonly, sandstones occur together with shales or siltstones as shown on Figures 3.23 and 3.24. In these situations weathering extends deeper into the mass and landsliding is more prevalent.

Figure 3.23, taken from Figure 13b of Deere and Patton (1971), shows a slope underlain by horizontally interbedded sandstones and shales. In high rainfall areas or

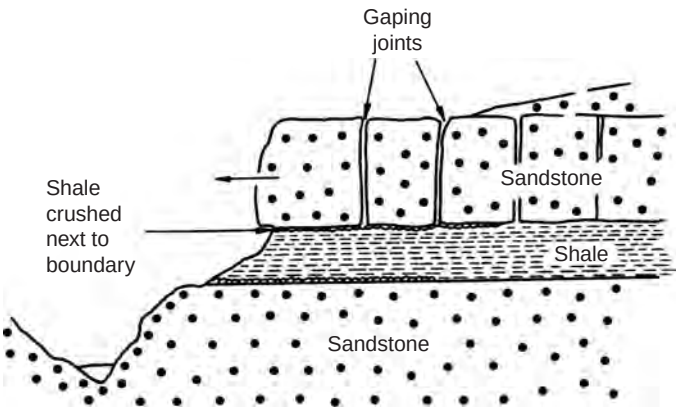


Figure 3.22a Commonly observed features close to cliffs formed by horizontally bedded sandstones with shale or siltstone interbeds. Based partly on Deere and Patton (1971).



Figure 3.22b Photographs of the abutment of Shannon Creek Dam, showing in-filled stress relief joints. The joints are up to 80 mm wide and terminate on shale beds. Photographs courtesy of L McDonald and Clarence Valley Council.

during the wet season in other areas, the water table on Figure 3.23 will be high and of the form shown, due to the relatively free-draining nature of the sandstone beds. Springs occur at the bases of sandstone outcrops and the shale below and between the sandstone beds is either continually wet or alternately wet and dry. Under these conditions chemical weathering of both rock types occurs but is usually more pronounced in the shale. This rock is at least partly weathered to clay and contains clay-coated joints or fissures, often slickensided. The weathered shale either slumps or its bearing capacity is exceeded, allowing large movements and eventual collapse of the outermost sandstone block. Continuation of these processes leads to development of layers

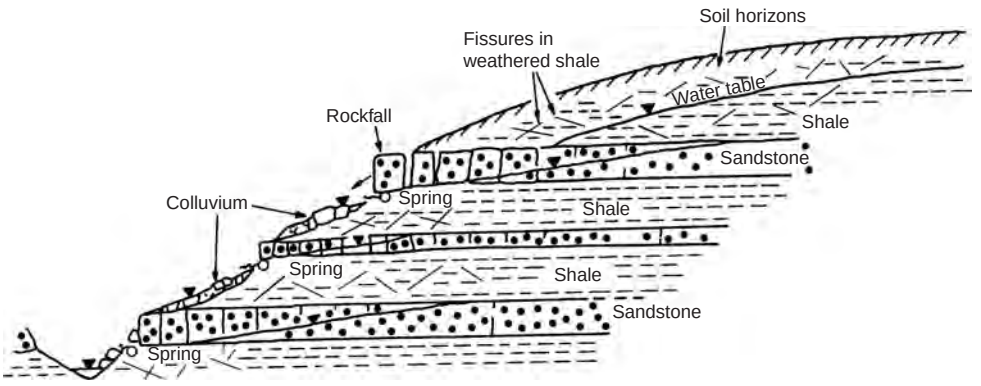


Figure 3.23 Collapse of outcropping sandstone beds due to removal of support of underlying shales (Based on Figure 13b of Deere and Patton, 1971).

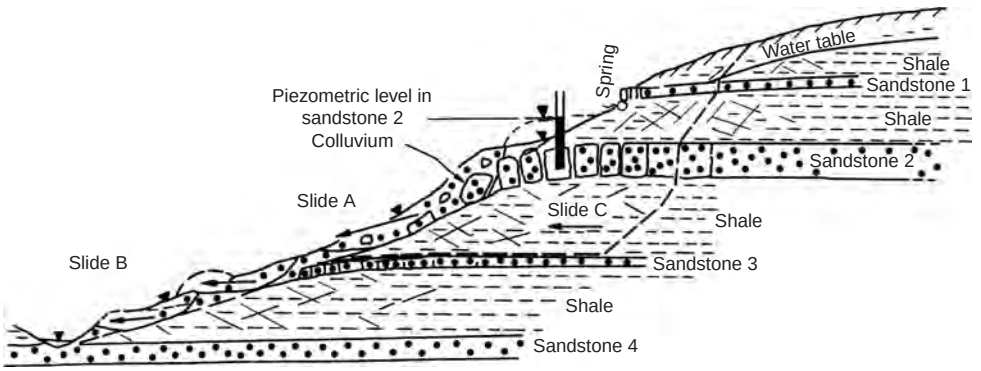


Figure 3.24 Landslides which can occur in a slope where sandstone beds are covered with colluvium (Based on Figure 13c of Deere and Patton, 1971).

of scree and colluvium on the slope, and to "cambering" of the near-surface part of the sandstone bed as shown on Figure 3.24.

Deere and Patton also describe landsliding observed commonly on interbedded sandstone/shale slopes where sandstone beds have become covered by colluvium, as shown on Figure 3.24. The colluvium restricts drainage from the sandstone (Bed 2) which may become a semi-confined aquifer with piezometric surface as indicated. Pore pressures so developed cause sliding to occur, usually along the colluvium-weathered shale contact (Slides A and B on Figure 3.24).

Slide B is a double slide in which the toe of the first slide (No. 1) has overloaded the meta-stable top of the slope below, causing Slide No. 2 (Deere and Patton 1971).

Deere and Patton (1971) point out that deepseated slides, such as Slide C on Figure 3.24, can occur when a combination of unfavourable geological conditions exists. These could include bedding surface crushed seams along the shale-sandstone boundaries, high water pressures in sandstones Numbers 2 and 3 and high water levels

in the affected mass. Deere and Patton point out that after the first small movements, the permeability of the deeper part of the mass will be increased and softening and weathering of the near-surface shale will be accelerated.

In folded sequences of sandstones with interbeds of siltstones or shales, landslides are relatively common where dipping beds daylight on steep slopes. Slides are also common on dipslopes. Examples of these types of sliding involving sandstones are given in Chapter 2, Sections 2.10.3.4 and 2.10.3.5.

3.6.5 Sandstones and similar rocks – list of questions

- Relatively high porosity, permeable?
- Gypsum or anhydrite present as cement?
- Quartzites: High quarrying and handling costs, difficult to compact?
- Rocks of medium or lower strength may not produce free-draining rockfill?
- Interbeds of shale or claystone?
- Bedding-surface faults at bed boundaries?
- Horizontal beds: Open joints and bedding surface crushed seams near surface due to stress relief?
- Horizontal beds with shale interbeds: Cambering and collapse due to removal of support by weathering shale?
- Landsliding in colluvium developed on weathering sandstone/shale slopes?

3.7 CARBONATE ROCKS

Carbonate rocks are defined here as those which contain significant amounts of the soluble minerals calcite, aragonite or dolomite in their substance fabrics. The most common are sedimentary carbonate rocks, marble (metamorphosed carbonate rock), and calc-silicate rocks formed by the metamorphism of impure carbonate rocks.

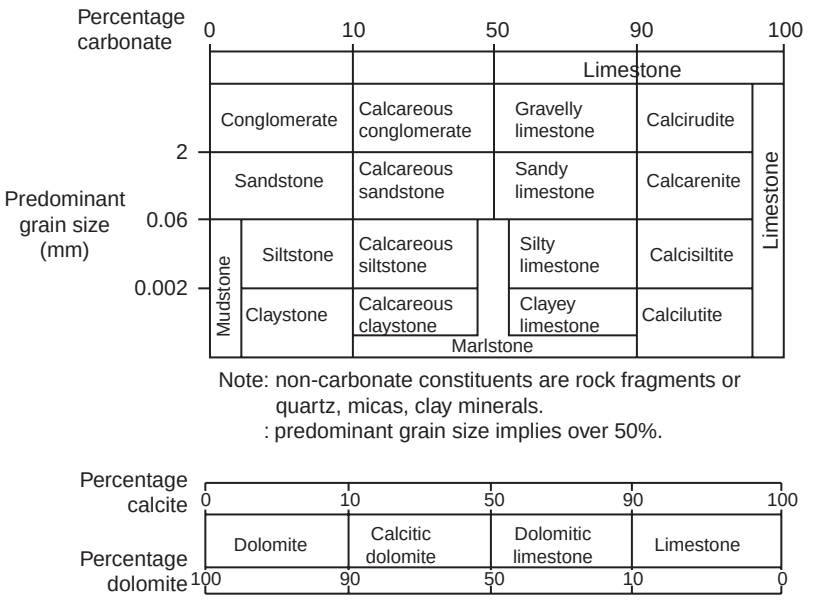
It is necessary to consider carbonate rocks in two categories, based on the dominant carbonate minerals present in each category.

Geologically young carbonate rocks (Category Y). Category Y carbonate rocks are usually of Tertiary, or younger, age. Most comprise loosely packed, weakly cemented shell fragments and are porous and weak to very weak. Rarely they can be well cemented and dense. The carbonate minerals are mostly aragonite and high-magnesian calcite (Friedman 1964, 1975; Molenaar and Venmans, 1993; Prothero and Schwab, 1996).

These two minerals formed all original marine carbonate sediments and are forming in present day deposits. There is field and laboratory evidence (see Sections 3.7.1.3 and 3.7.7.2) that high-magnesian calcite is more susceptible to dissolution and cementation than aragonite and calcite. With time, exposure to fresh water, compaction and recrystallisation both aragonite and high-magnesian calcite eventually revert to calcite, and the rock becomes *Category O*.

Geologically old carbonate rocks (Category O). Category O carbonate rocks are generally of Mesozoic or older age and are usually dense, non porous and range from strong to extremely strong. They are formed by the minerals calcite or dolomite. They include marble, which comprises coarsely crystalline calcite and is usually dense, non

Table 3.2 Engineering classification of sedimentary carbonate rocks (Dearman, 1981).



porous and strong to very strong. Also included are calc-silicate rocks, which contain carbonate minerals together with silicate minerals often including olivine, diopside and garnet.

The carbonate rocks at the sites of most large dams are *Category O*. The exceptions mentioned in this book are Kopili Dam, Perdikas, Montejagne and May Dams, each of which failed to store water (Table 3.4).

Table 3.2 proposed by Dearman (1981) is a practical engineering classification for sedimentary carbonate rocks. Dearman (1981), Cruden and Hu (1988) and Bell (1981, 1983a and 1992) present data on physical properties of some carbonate rocks. Fookes and Higginbottom (1975) provide a table which classifies and names various types of original pure carbonate sediments (detrital, chemical and biochemical) and their progressively strengthened counterparts. This table has useful descriptive terms and its accompanying discussion indicates some of the complexities involved in the strengthening of carbonates.

Calcrete (caliche) is a *Category Y* carbonate rock of highly variable strength found in many arid areas. The most common variety, pedogenic calcrete, occurs within a metre or so of the ground surface (Netterberg, 1969, 1971; James and Choquette, 1984; Meyer, 1997).

3.7.1 Effects of solution

Solution is one of the processes of chemical weathering which affects all rock types to some extent. It is more severe and causes cavities in carbonate rocks because calcite



Figure 3.25 Karst landscape in New Zealand. The outcrops of limestone are up to 1.5 m high and separated by soil-covered holes and slots up to 1 m across.

and dolomite are relatively soluble in acidic waters, e.g. carbonic (dissolved carbon dioxide), organic (from vegetation) or sulphuric (from volcanic activity or oxidation of sulphides). They are more soluble in saline water than in pure water and their solubility increases with desceasing water temperatures. Also, their rates of solution are high (James and Kirkpatrick, 1980; James, 1981, 1992).

The term karst is used to describe terrain underlain by cavernous rocks (usually carbonates or evaporites) and also to describe the cavernous rocks themselves. The factors involved in the formation of karst are the same as those which contribute to the development of weathered profiles described in Chapter 2, Section 2.6.4. Detailed discussion of karst development, structure and hydrogeology can be found in Sweeting (1972, 1981), Milanovic (1981), Bonacci (1987), Ford and Williams (1989), Beck (1993, 1995) and De Bruyn and Bell (2001).

The effects of solution as seen in the most common types of carbonate rock masses are discussed below.

3.7.1.1 *Rock masses composed of dense, fine grained rock substances comprising more than 90% of carbonate (usually Category O)*

When fresh and intact these rocks usually have very low porosities and their substance permeabilities are effectively zero. Groundwater flow is confined to joints or other defects and these are widened by solution to form slots, shafts, tunnels and cavities of all shapes and sizes. Important features are:

– The upper surface of rock, both in outcrops and below ground, usually consists of pinnacles of fresh rock separated by deep slots or cavities, which may be empty or soil-filled (Figures 3.25 and 3.26).



Figure 3.26 Close up view of limestone pinnacles.

- No weathered rock substance is present, either at the ground surface or at depth. This is because dissolution of the relatively pure limestone substance leaves less than 10% of insolubles, which form residual soils at or near the ground surface and infill some cavities below (Figure 3.27). The residual soils are commonly fissured and rich in clay and iron oxides.
- Dam construction experience has shown that cavities capable of accepting or transmitting large inflows of water can be met at any depth down to 300 m below ground surface (see Section 3.7.2.1).
- Sinkholes, usually shaft-like cavities, are often exposed at the ground surface and the formation of new sinkholes can be initiated by natural or man-made activities.

Figure 3.27 represents the uppermost 10 m or so of a karst profile beneath a small area showing relatively small cavities and solution-widened defects, which would be connected to larger cavities at greater depths. It also illustrates some of the mechanisms by which sinkholes have been formed.

Feature (a) is a sinkhole formed directly by the collapse of the roof of a cavity.

Feature (b) can also be referred to as a sinkhole. It is a gentle, soil-covered depression, roughly circular in plan view. Exploration shows it to be underlain by residual

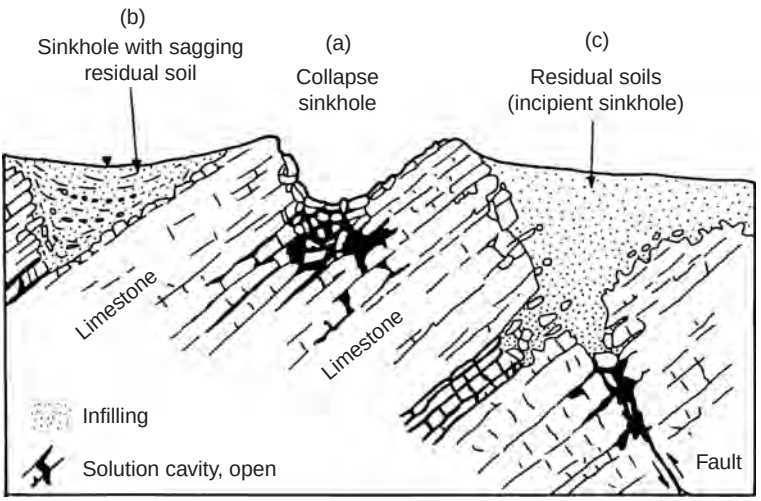


Figure 3.27 A typical weathering profile in dense, relatively pure carbonate rock (Based on Deere and Patton, 1971).

soil with a concave upwards layered structure. It can be inferred that this sinkhole has developed because the residual soil has migrated slowly into the cavities in the limestone below.

Feature (c) is a large flat area showing residual soil at the surface. It is underlain at depth by cavities developed along joints and a fault zone. Such flat areas may show no obvious evidence of settlement or disturbance, but there is a significant risk of a deep, steep-sided sinkhole forming (with little or no warning) at the ground surface. This could be initiated by migration of soil into the cavities below, causing a large void to develop near the base of the residual soil. The sinkhole could then form by collapse of the overlying soil into the void. A similar dangerous sinkhole could also develop within the gently sloping sinkhole (b).

The mechanisms of sinkhole formation and their significance in dam engineering are discussed further in Section 3.7.3.

3.7.1.2 *Rock masses composed of dense fine grained rock substance containing 10% to 90% of carbonate (usually Category O)*

In these rocks the pattern of solution cavities is similar to that described in Section 3.7.1.1 and seen in Figure 3.27, but usually the rock substance next to some of the cavities is weathered. Experience and logic show that in general the lower the percentage of carbonate in the fresh rock the higher the proportion of weathered rock formed compared to cavities and the higher the ratio of infilled cavities to open cavities. The weathered rock is usually much weaker and less dense than the fresh rock. However, the proportion and properties of weathered rock formed depend also on the percentage of insoluble particles present in the fresh rock and the degree to which these particles are bound by non-soluble cements. These effects are illustrated by strong dolomitic



Figure 3.28 Weathering profile on very weak, porous calcarenite near Perth, Western Australia. Note the extremely irregular nature of the rock surface.

siltstones which occur widely in South Australia. Many weathered exposures of these rocks appear to have lost all of their dolomitic cement, but are still weak or medium strong rock, due to some non-carbonate cement. A few beds within such a sequence at and near Little Para Dam appear to lack this non-soluble cement and these beds form very low density, very weak rock, grading to clayey silt and small cavities (see Figure 3.33 in Fell *et al.*, 1992).

3.7.1.3 *Rock masses composed of porous, low density carbonate rock substance (usually Category Y)*

Most of these rocks are weak to very weak calcarenites. In many cases the carbonate grains are recognizable shell fragments. The rocks are usually in horizontal or gently dipping beds and are relatively free from joints or other defects of tectonic origin.

These rocks have sufficient substance permeability to allow surface waters to enter and migrate downwards, causing both solution and redeposition effects. The 8 m high quarry face in Figure 3.28 shows a typical profile. The very weak calcarenite here is a windblown deposit of Pleistocene Age. It is off-white in colour and overlain by yellow-brown quartz which appears dark grey in the photograph. The profile in the calcarenite is characterized by the following:

– Vertical pipes or tubes; each is surrounded by an annular zone of calcarenite which is much more dense and strong than elsewhere in the mass. The tubes have been infilled with the quartz sand, so show up well where exposed on the face.
– Vertical pinnacles of calcarenite which are much more dense and strong than elsewhere in the mass.

The tubes have been formed by solution of the calcarenite. Strengthening of their rocky surrounds and of the pinnacles appears to have been caused by redeposition of calcite derived from that solution process and possibly also from solution of calcarenite which previously existed above the present ground surface.

The porous, low-density carbonates rarely contain large cavities like those in dense, jointed carbonates. However Twidale and Bourne (2000) describe many shallow sinkholes up to 10 m across, and several much larger, on wind-deposited calcarenite in South Australia. They present evidence which suggests that these have developed due to solution along localized calcarenite zones rendered more permeable by small movements along faults in the bedrock at a depth of 100 m or more.

Chalk is very low strength, low-density calcilutite (Table 3.2, and Table 13.8 in Bell (1983b). It has extremely low substance permeability, but can be permeable where joints are present. Bell (1983a) and Goodman (1993) point out that large cavities are rare in chalk, but solution pipes and sinkholes can occur, near its contact with colluvium or other rocks.

3.7.2 Watertightness of dam foundations

Many dams have been built successfully on sites underlain by *Category O* carbonate rocks, despite the fact that solution cavities have been present at most sites and beneath at least part of their storage areas.

Selection of dam foundation levels and treatments can be difficult, due to the highly irregular nature of the uppermost surface of the fresh rock, as shown by Figures 3.25 to 3.28. This applies particularly to embankment dams, because of their large foundation areas and the need to provide stable, non-erodible surfaces for placement of embankment materials.

Treatments to fill cavities and prevent excessive leakage have included cement grouting, concrete curtain walls and selective mining and backfilling with concrete. The presence of clay infilling in cavities presents a problem when cement grouting is proposed. Clay usually prevents grout penetration and is not readily removed by flushing between boreholes. However, if left in place, it may be eroded out later when the dam is filled. High pressure cement grouting designed to cause hydraulic fracturing of clayey infills has been shown by Zhang and Huo (1982) to give significant improvement in their resistance to leakage and piping.

At the 35 m high Bjelke-Peterson earth and rockfill dam in Queensland about 250 m of the left bank was formed by a landslide deposit underlain by limestone. In this section the cutoff trench was excavated down to the water table, 15–20 m below the original surface and 5–15 m below the irregular upper surface of the limestone (See Chapter 10, Figure 10.8). At the trench base the limestone was mainly fresh, dense and very strong, but contained open and infilled solution cavities within and next to sheared and crushed zones (Eadie, 1986; McMahon, 1986). One of these zones was locally deepened by up to 8.5 m and backfilled with concrete. Throughout the limestone sections the base of the trench was covered by a reinforced concrete slab anchored into the rock. Cavities exposed on the downstream face of the trench were sealed with concrete. A single row grout curtain below this grout cap used 32 m deep vertical holes at 0.5 m spacing. Each 8 m stage, before testing and grouting, was flushed by high pressure air and water until the wash water was substantially clear. McMahon

Table 3.3 Dams at which very high leakage rates were recorded (Based on Erguvanli, 1979; Riemer *et al.*, 1997; Salembier *et al.*, 1998).

Dam	*Country*	*Rock*	*Maximum recorded leakage (m^3/s)*
Hales Bar	USA	Carboniferous limestone and shale	54
Keban	Turkey	Palaeozoic limestone	26
Lar	Iran	Mesozoic limestone, volcanics, lake deposits	16
Attaturk	Turkey	Palaeozoic limestones, folded and faulted	14
Great Falls	USA	Limestone	13
Camarasa	Spain	Jurassic dolomitic limestone	12
Dokan	Iraq	Cretaceous dolomitic limestone	4 to 5
Fodda	Morroco	Jurassic limestone	3 to 5

(1986) believes that this washing was unsuccessful. Relatively high grout pressures were used (3 times the reservoir head), aimed at achieving hydraulic fracturing of the clayey infill materials, like that reported by Zhang and Huo (1982).

The general foundation levels for the rockfill shoulders were reached by scrapers. In the limestone areas the exposed rock was essentially strong mounds and pinnacles separated by clay-filled depressions. These were cleaned out to the depth at which the clay filling was less than 1 m wide and backfilled with high quality rockfill.

At sites where cavities are numerous and/or large and partly or wholly filled with clayey soils, cement grouting alone is not usually relied on to form the cutoff. Other methods have included:

- Cleaning out of some individual caves and backfilling with concrete,
- Walls formed by mining out slots of cavernous rock and backfilling with concrete,
- Diaphragm walls comprising overlapping boreholes backfilled with concrete,
- Closely spaced drilled holes, washed out with compressed air and water and backfilled with high-slump mortar, poured in and needle vibrated.

The last three methods were used, together with cement grouting, at the 85 m high, 1050 m long concrete face rockfill Khao Laem dam in Western Thailand. At the dam they were used to construct the upper 20–60 m of a curtain which extended up to 200 m (but generally less than 100 m) below the plinth. In one section, high pressure flushing with air and water was used to flush out sandy silt (weathered calcareous sandstone) from cavities and open joints before grouting.

Another curtain, 3.5 km long and up to 200 m deep, was constructed beneath the right abutment ridge, which was formed entirely by cavernous limestone, with its water table below the proposed storage level. All four methods were used in parts of this curtain, together with cement grouting. Somkuan and Coles (1985) describe details of the methods.

3.7.2.1 *Dams which have experienced significant leakage problems*

Table 3.3 lists dams which have recorded very high leakages, and Table 3.4 lists others which have never stored water because the leakage rates exceeded the inflow. Two dams which have recorded very high leakages are discussed below.

Table 3.4 Dams which failed to store water (Erguvanli, 1979).

Dam	*Country*	*Rock*
Civitella Liciana	Italy	Cretaceous limestone
Cuber	Spain	–
Kopili	India	Eocene limestone*
May	Turkey	Mesozoic and Tertiary limestone*
Montejagne	Spain	Mesozoic and Tertiary limestone*
Perdikas	Greece	Miocene limestone*
Villette Berra	Italy	–

*Geologically young carbonate rock (*Category Y*) present.

Attaturk Dam. Riemer *et al.* (1997) describe cavities met during the planning and construction of this 179 m high, 1800 m long rockfill dam in Turkey, completed in 1990. The site is formed by a folded sequence of *Category O* limestones including marly, cherty and bituminous units. The rock substances are moderately strong to weak, dense and impermeable. The rock mass is cut through by many steeply dipping faults and joints. Solution along these has resulted in a network of chimneys and near-horizontal tunnels.

Despite the use of many adits and drill holes, the design stage studies failed to indicate the magnitude of this cavity network and of the grouting program required.

The main 3-row grout curtain is 5.5 km long and extends generally 175 m below the foundation with a local extension down to 300 m. Even at these depths the curtain is described by Riemer *et al.* (1997) as "suspended". An elaborate monitoring program involving 3.6 km of adits and about 300 piezometers has indicated seepage discharge rates are governed mainly by reservoir level. The (1997) total discharge is estimated as 11–14 m^3/sec, with reservoir levels ranging from 6–8 m below FSL. These losses are tolerable, as the average flow of the impounded river is 850 m^3/sec. The monitoring also shows some high piezometric levels downstream from the curtain and some decrease in head upstream of it. Riemer *et al.* (1997) point out that these results "could hint at" ongoing erosion, either of grout or clayey infilling and they foreshadow the possible need for further grouting and/or drainage works.

Lar Dam. This is an earthfill embankment dam, 110 m high and 1100 m long, across the valley of the Lar River in Iran. Djalaly (1988) and Salembier *et al.* (1998) summarise the site geology and describe difficulties met since completion of the dam construction in 1980. Uromeihy (2000) provides more detail on the geology. The site is at the foot of an active volcano. Bedrock in the area is complexly folded, faulted and karstic limestones and mudrocks of Mesozoic Age. The dam and 15 km long reservoir area are located partly on these rocks and partly on near-horizontal materials of Quaternary Age. These include lava, ash, alluvial and lake deposits (mainly sandy and silty). They fill an old valley, which is around 200 m deep at the dam site and up to 600 m deep in the reservoir area. Before construction there were several large sinkholes (up to 25 m across and 30 m deep) in the alluvial/lake deposits in the reservoir area. The water table in the dam area was more than 200 m below the river bed and there was a karstic spring 10 km downstream from the dam, flowing at about 0.5 m^3/sec.

Uromeihy (2000) notes that outgassing CO_2 and H_2S from hot springs flanking the volcano would increase the acidity of the streams draining into the reservoir but provides no data on the chemistry of the reservoir water.

After impoundment started in 1981, large new sinkholes appeared in alluvium. These were exposed when the lake was drawn down and many depressions in the lake floor were located by sonar survey. Uromeihy (2000) showed that 8 large sinkholes within 6 km upstream from the dam lie close to the trace of a known fault. The water table near the dam rose by 80–100 m. New springs appeared downstream and flow from the original spring increased to 9.9 m^3/sec. The total flow rate from all springs indicated that leakage from the reservoir was about 16 m^3/sec. This was more than the average inflow at the site and the highest reservoir level reached was 23 m below Full Supply Level. After studies and analyses of the leakage, site exploration, grouting and other remedial works were commenced in 1983. These involved 55,000 m of drilling and 100,000 tonnes of injected materials. The largest cavern located and treated was 200 m below the river bed, and was 27 m high and 68 m wide. Smaller cavities were located down to 430 m below the river bed (Djalaly, 1988). The treatment works did not lower the leakage rate and were stopped in 1990.

In 1991 site studies started again, but with a new emphasis, as indicated in the following words by Salambier *et al.* (1998) "*The key to understanding the leakage conditions and consequently proposing the most appropriate measures was geological modelling based on extensive investigation and comprehension of the geological history of the site*".

The scope of the new studies is summarized by Salambier *et al.* (1998). They included geological surface mapping, interpretation of air photographs and satellite imagery, an extensive geophysical program and drilling 10 holes totalling 3,750 m. The deepest drill hole was 650 m.

This work resulted in improved geological and hydraulic models from which Salambier *et al.* (1998) showed that the extent of permeable ground was much greater than previously assumed and that there was a serious problem with potential sinkhole formation (see Section 3.7.3).

They also predicted the extent of ground requiring treatment to solve the leakage and associated problems and were able to delineate this in plan view and on sections. In the bedrock beneath the dam and its right abutment the treatment area continued to 600 m below river bed level as shown on Figure 3.29. Under the storage area the treatment area was shallower, but extended more than 5 km upstream from the dam.

Salambier *et al.* (1998) concluded that "*successful plugging and grouting treatment cannot be guaranteed*" and that the cost could not be estimated "*within a reasonable range of certainty*". They recommended that, as an alternative, blanketing of the ground surface be considered – by shotcrete and poured concrete on the limestone and by geomembrane, over the alluvial/lake deposits. Clay was considered feasible, but too scarce near the site. Reimer (personal communication) advises that as at mid-2003 no work had been done.

3.7.3 Potential for sinkholes to develop beneath a dam, reservoir or associated works

Sinkholes which develop naturally (Figure 3.27) are present in most areas underlain by carbonate rocks. In many such areas collapse of the ground surface to form a new sinkhole is a relatively common occurrence. Some of these collapses may happen naturally but more often they are induced or, accelerated by man's

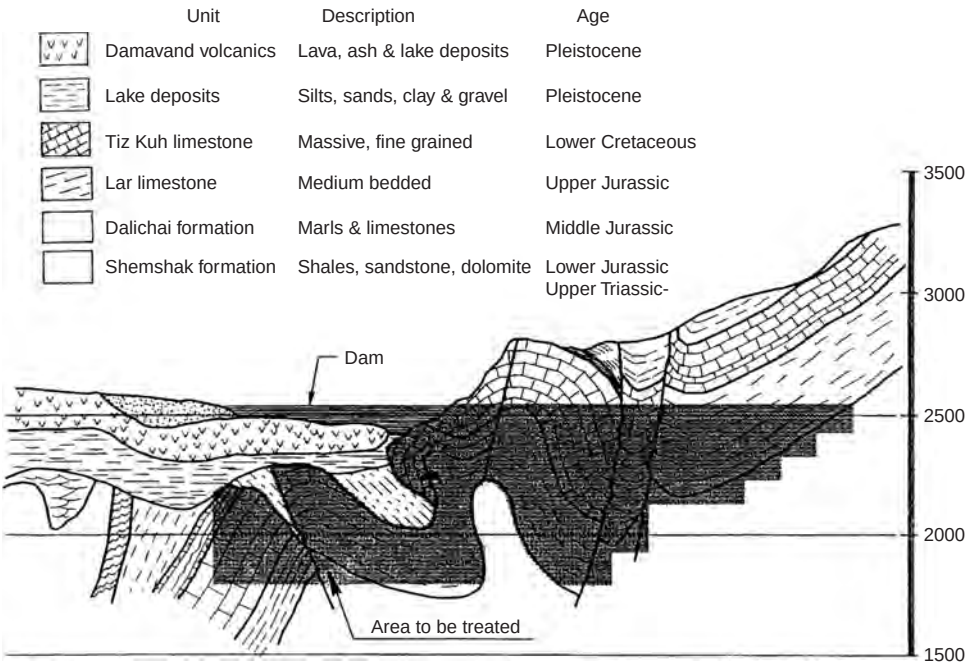


Figure 3.29 Lar Dam, cross section showing the predicted extent of ground treatment needed at the dam site (Salambier *et al.*, 1998).

activities, as seen at Lar Dam. Possible mechanisms for sinkhole formation include the following:

1. *Dewatering*. This can cause loss of buoyant support of the rock or soil forming the roof of a cavity or, by steepening the hydraulic gradient, cause erosion and collapse of overlying soils into the cavity. It may also cause collapse due to drying out, causing shrinkage and ravelling failure of overlying soils.
2. *Inundation*. This can cause previously dry soil to lose apparent cohesion and collapse or erode into a cavity or can steepen the gradient in previously wet soils, causing them to erode into a cavity or into joints widened by solution.
3. Vibrations from machinery or blasting.

Examples of most of the above have been described by Sowers (1975), Brink (1979), Newton and Turner (1987) and Wilson and Beck (1988).

Chen *et al.* (1996) have suggested that a rapidly rising water table in cavernous rock could cause a "water hammer effect" – pressure waves which might accelerate or induce collapse of overlying soils into cavities. First and subsequent fillings of a reservoir could provide these rapid rises in a water table.

It is clear that for all sites in karst areas there will be some risk of ground surface collapse affecting any part of the project works, including borrow areas and haul roads as well as the dam embankment and associated works. The worst imaginable event

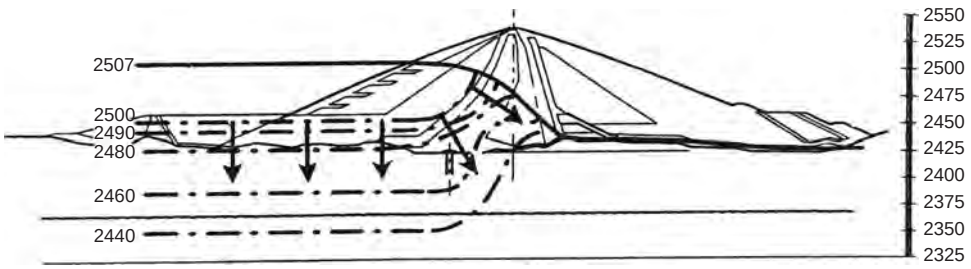


Figure 3.30 Lar Dam, flow net in the embankment and foundation (Salambier *et al*., 1998).

of this type would be a major sinkhole forming beneath a dam embankment during reservoir operation.

The possibility of this situation arising at Lar Dam has been accepted by Salambier *et al*. (1998). The dam has been extensively monitored, with instruments including standpipe, hydraulic and pneumatic piezometers. From a flow net for the dam and its foundation (Figure 3.30) they point out that:

– Vertical gradients exist in the upstream part of the core, in the upstream transition zone and in the alluvial foundation and
– Piping of any of these materials is possible if the reservoir is operated at higher levels.

They conclude that "*... development of a large sinkhole cannot be excluded. If this were to happen, complete failure of the embankment could occur*".

Uromeihy (2000) warns that there is a risk of sinkhole development also beneath the downstream shoulder

3.7.4 Potential for continuing dissolution of jointed carbonate rock in dam foundations

The following sections relating to dissolution processes and effects rely heavily on theoretical and experimental material described by James and Lupton (1978, 1985), James (1981) and James and Kirkpatrick (1980).

Most of the material in those papers is included and expanded upon in James (1992), a comprehensive book covering a wide range of aspects ranging from the fundamentals of solubility to the engineering behaviour of all soluble rocks, i.e. carbonates and evaporites. Also included are useful case histories of dams and other structures in which solution effects were investigated.

James and Lupton (1978) discuss the principles governing the dissolution of minerals, develop mathematical models which predict how the minerals gypsum and anhydrite dissolve in the ground and describe laboratory tests which confirm the validity of the predictions. Water flow through both jointed rock and porous granular

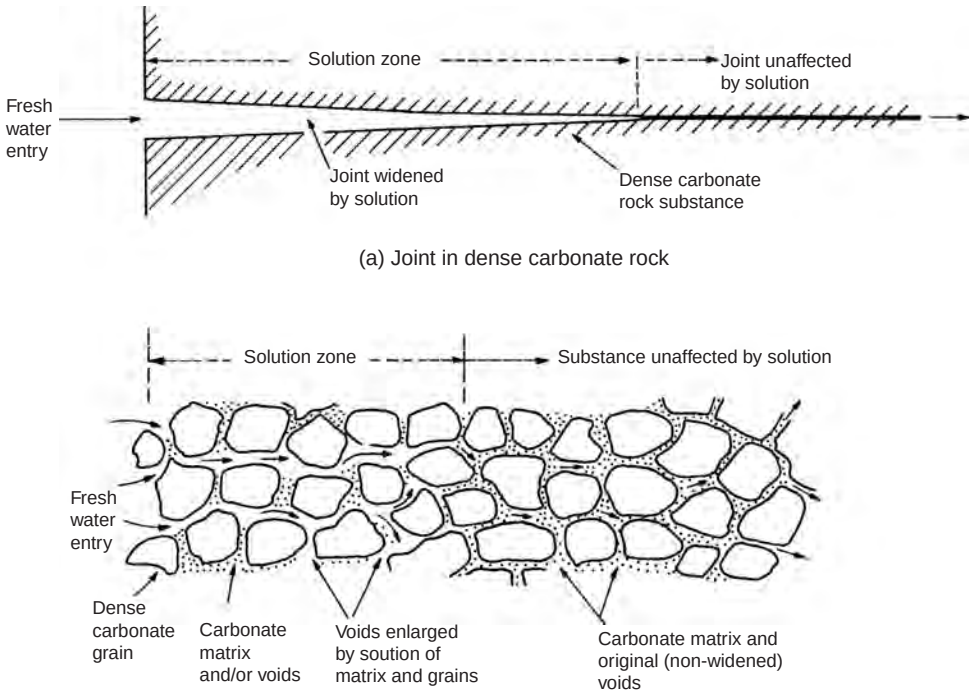

Solution zone
Joint unaffected by solution
Fresh water entry
Joint widened by solution
Dense carbonate rock substance
(a) Joint in dense carbonate rock
Solution zone
Substance unaffected by solution
Fresh water entry
Dense carbonate grain
Carbonate matrix and/or voids
Voids enlarged by soution of matrix and grains
Carbonate matrix and original (non-widened) voids
(b) Permeable carbonate rock substance, or aggregate of carbonate particles

Figure 3.31 Development of solution zones in (a) a joint and (b) soluble rock substance or aggregate of carbonate particles (Based on James and Lupton 1978).

material are considered. They show that the rate at which the surface of a mineral or rock retreats depends primarily upon two properties of the mineral, namely:

- the solubility (c) of the mineral, which is the amount which can be dissolved in a given quantity of solvent, at equilibrium and
- the rate of solution of the mineral, which is the speed at which it reaches the equilibrium concentration.

The solution rate constant (K) is further dependent on the flow velocity and temperature of the solvent and concentrations of other dissolved salts.

James and Lupton (1978) point out that for water flowing through a joint in an effectively impervious but soluble rock substance, widening of the joint by solution of the rock walls will occur only until the water becomes saturated with the soluble material. Thus a solution zone (Figure 3.31a) is initiated near the fresh water interface and migrates progressively downstream. For water flow through soluble rock substance with intergranular (fabric) permeability a comparable solution zone is formed and migrates downstream (Figure 3.31b).

James and Kirkpatrick (1980) use the methods and models of James and Lupton (1978) to derive conclusions on foundation treatments needed to prevent dangerous progressive dissolution when dams are built on rocks containing gypsum, anhydrite, halite and limestone. They use the following solution rate equations to predict the ways in which the materials dissolve:

For gypsum, halite and limestone

$$\frac{dM}{dt} = KA(c_S - c) \tag{3.1}$$

For anhydrite

$$\frac{dM}{dt} = KA(c_S - c)^2 \tag{3.2}$$

where M is the mass dissolved in time t, A is the area exposed to solution, c_S is the solubility of the material, c is the concentration of material in solution at time t and K is the solution rate constant.

The limestone used in their laboratory tests was Portland stone, a dense, old rock (Jurassic Age), in which the carbonate mineral is calcite. For this rock they showed that K values rose sharply at flow rates close to the onset of turbulence. They note that these "transitional" velocities (around 0.75–0.8 m/s) would be reached by water flow in joints with apertures of about 2.5 mm, with an hydraulic gradient of 0.2.

James and Kirkpatrick (1980) use mathematical modelling to provide predictions of the way joints enlarge as water flows through them. Their Figure 3, reproduced here as Figure 3.32, shows their predictions for the enlargement by pure water of limestone joints with initial apertures ranging from 0.5 mm to 2 mm.

It can be seen that the solution occurs essentially by downstream migration of the solution zone, with no enlargement further downstream. After a period of 100 years, the solution zone of the 0.5 mm open joint has migrated only 13 m from the inlet face. The 0.7 mm joint shows a similar type of behaviour, but the enlarged portion migrates 30 m in only 40 years. During very short periods the larger aperture joints show long tapered enlargements indicating that in real dam situations (i.e. seepage paths often much less than 100 m) seepage flows would accelerate.

James and Kirkpatrick (1980) conclude that the smallest aperture joint which will cause dangerous progressive solution in limestone is 0.5 mm for pure water or 0.4 mm for water containing 300 mg/l of dissolved carbon dioxide. They further conclude that if all large cavities are backfilled with grout or concrete, then cement grouting (which can fill joints down to about 0.2 mm aperture) should be adequate to prevent progressive solution of limestone foundations. They point out the need for care in the conducting of Lugeon permeability tests and in estimating the apertures of joints from the results of the tests.

Table 3.5 summarises the conclusions of James and Kirkpatrick (1980) for preventing progressive solution along joints in all four soluble rock types.

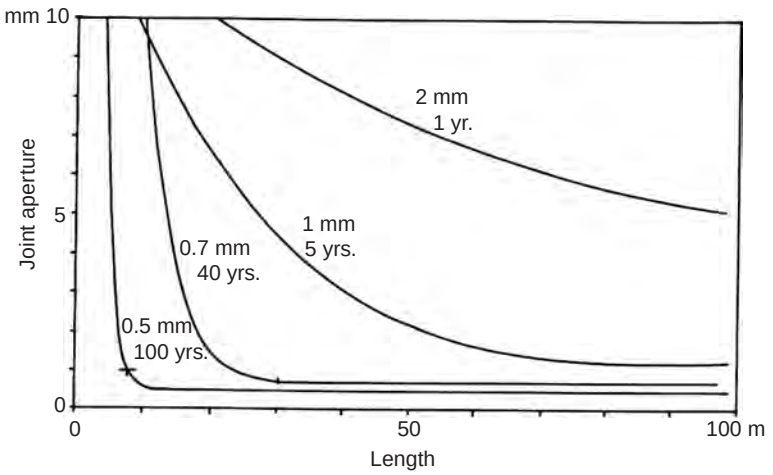


Figure 3.32 Enlargement of joints in dense *Category O* limestone (Portland stone) by pure flowing water (James and Kirkpatrick, 1980, reproduced by permission of the Geological Society).

Table 3.5 Solution of jointed rock (from Table 3 of James and Kirkpatrick, 1980).

	Largest joint aperture (mm)		
Mineral	*For stable inlet face retreat*	*For rate of retreat of 0.1 m/year*	*Suggested preventive measures*
Gypsum	0.2	0.3	Grouting
Anhydrite	0.1	0.2	Cut-off – e.g. cement-bentonite
Halite	0.05	0.05	Cut-off – e.g. cement-bentonite
Limestone (Category O)	0.5	1.5	Grouting

Note: Values are for pure water; at joint spacing of one per metre and a hydraulic gradient of 0.2.

3.7.5 Potential for continuing dissolution of aggregates of carbonate rock particles and of permeable carbonate substances (Category O carbonate, in each case)

James and Kirkpatrick (1980) also discuss solution rates in "particulate deposits". It is assumed that such deposits might comprise either aggregates of rock fragments (as in some zones of sheared or crushed rock, or in filter zones) or substances rendered permeable by intergranular voids (Figure 3.31b). Using the theoretical models of James and Lupton (1978), James and Kirkpatrick show that the length of the solution zone depends mainly on the solution rate constant and the solubility and that other factors such as the shape and sizes of the particles and the proportion of soluble minerals present, are of secondary importance. They show further that the downstream migration rate of the solution zone is governed largely by the seepage velocity and by the solubility, the proportion of soluble mineral present being less critical.

Table 3.6 Solution of particulate mineral deposits (James and Kirkpatrick, 1980).

Mineral	*Limiting seepage velocity (m/s)*	*Length of solution zone (m)*
Gypsum	1.4×10^{-6}	0.04
Anhydrite	1.6×10^{-6}	0.09
Halite	6.0×10^{-9}	0.002
Limestone (Category O)	3.0×10^{-4}	2.8

Note: Rate of movement of solution zone is 0.1 m/year. Mineral particles diameter 50 mm. Pure water.

Table 3.6 of James and Kirkpatrick (1980), reproduced here as Table 3.6, shows the calculated seepage velocities which would be required to cause 0.1 m/year downstream migration of solution zones in a range of such "particulate mineral deposits". Based on their opinion that the required seepage rate for "particulate" limestone (3.0×10^{-4} m/s) is so high that it would be "rarely tolerated in dam foundations" they conclude that control of seepage by a good grouting program or other means should prevent dangerous progressive solution of "particulate" forms of dense carbonate rock.

James (1981) determined the solution parameters for 10 carbonate rocks of different compositions – 8 limestones, 1 calcitic dolomite and 1 probable chalk. They ranged in age from Carboniferous to Cretaceous. Using finely crushed samples and pure water, he found their solubilities to be virtually the same as that of "pure calcium carbonate" (presumably calcite) and that differences in their solution rates were too small to be significant in engineering design.

James (1981) showed for one sample of limestone, that small amounts of carbon dioxide dissolved in the water lowered the solution rate by about a factor of 10, but caused large increases in the solubility. Under "Engineering considerations" he confirmed the conclusions and recommendations of James and Kirkpatrick (1980), but recommended that the following were also needed when assessing the potential for progressive solution of carbonate rock in a dam foundation:

– Chemical analyses of the appropriate reservoir, river or groundwaters and
– Laboratory tests to determine the solubility of the carbonate rock in these waters.

3.7.6 Discussion – potential for continuing dissolution of carbonate rocks in foundations

3.7.6.1 *Category O carbonate rocks*

A conclusion which might be drawn from the work of James and Kirkpatrick (1980) and James (1981) as discussed in Sections 3.7.4 and 3.7.5, is that dangerous ongoing solution of old, dense carbonate rocks in a dam foundation can always be avoided by a thoroughly planned and executed grouting program using Portland cement. However, early in their paper, under "Modelling of ground conditions", James and Kirkpatrick include the following warnings against such a general conclusion.

"The conditions analysed below are idealized and do not take account of complex variations which occur in the ground" and

"Eventually it must be a matter for the engineer to judge what reliance to place on site investigation data and other engineering geology considerations, and thence to decide what factors of safety to employ".

The authors note that the individual conclusions of James and Kirkpatrick (1980) and James (1981) assume:

(a) the relatively simple joint and "particulate" models show on Figure 3.31,
(b) the largest joint apertures for limestone, on Table 3.5 and
(c) the limiting seepage velocity of 3×10^{-4} m/s for pure water through particulate or fragmented limestone, on Table 3.6.

With regard to (a) and (b) we note that at sites in faulted and weathered carbonate rocks the geological situation as exposed in detail at the excavated foundation surface and in tunnels, shafts or boreholes below it, can be very complex. In addition to features of the types on Figure 3.31, any or all of the following have been observed, often at great depths below ground surface:

- Open cavities or tunnels of all sizes, regular or irregular shapes, connected or disconnected,
- Cavities as above, partly or wholly filled with soils of variable erodibility,
- Beds or masses of variably weathered and erodible carbonate rock (see Section 3.7.1.2) and
- Beds or masses of weathered and erodible non-carbonate rock.

Achieving a satisfactory grout curtain can be extremely difficult or impossible in a foundation containing such features. During dam operation removal of soil or weathered rock by erosion could provide a flow path wide enough to cause ongoing solution of adjacent limestone. This type of process may have occurred at Hales Bar, Lar and Attaturk Dams (see Section 3.7.2.1 and Table 3.3).

With regard to (c), the maximum leakage rates shown on Table 3.3 suggest that seepage velocities much higher than 3×10^{-4} m/s are/were being tolerated, at least locally, through the foundations of some of these dams. Such local flows may have high velocities and could be turbulent. It is noted also that Bozovic *et al.* (1981) and Riemer *et al.* (1997) foreshadow the possibility of future increases in leakages at Keban and Attaturk Dams.

However, as discussed previously in Section 3.7.2, many dams have been built in *Category O* limestone sites, with cement grouting for seepage control and have performed well for many years without excessive or increasing seepage. The authors accept that a good grouting program should be adequate to control seepage and prevent ongoing solution at many dams built on such carbonate rocks. However, it is recommended that those responsible for building any dam on these rocks should:

- Pay special attention to the design of monitoring systems (see Chapter 20) and
- Make adequate allowance (e.g. access and ability to lower storage levels) for additional grouting programs which might be required after first filling or later during the lives of the dams.

3.7.6.2 *Category Y carbonate rocks*

The resistance of *Category Y* carbonate rocks to dissolution in fresh water has not been researched to the extent of the work done on *Category O* rocks by James and Kirkpatrick (1980) and James (1981). However, the following reasons suggest strongly that their potential for continuing dissolution, if present in a dam foundation, would be higher than that for *Category O* rocks:

- Their lower strengths and higher porosities,
- The field and laboratory evidence of their rapid solution and redeposition, presented in Section 3.7.7.2 and
- The experiences at Kopili, Montejagne, Perdikas and May dams (Table 3.4).

3.7.7 Potential problems with filters' composed of carbonate rocks

Most filters are well graded sands or gravels with few or no fines. They are compacted to 60% to 70% density ratio. Their moisture environments (in chimney zones, in particular) may vary widely. For example, in a filter, months or years of inundation and large or small flow rates may be followed by months or years of unsaturated, ranging to almost dry, conditions. Heavy rainfall at any time can cause water containing carbon dioxide to enter the upper part of a chimney zone. Carbon dioxide developed by the oxidation of methane from the groundwater or reservoir, organic acids derived from rotting vegetation or sulphuric acid derived from oxidation of sulphide minerals may also enter a filter zone. Possible effects of these events on a carbonate rock filter include:

(a) change of grading due to dissolution or
(b) partial dissolution and recementation or
(c) interlocking of grains due to pressure solution (see Figure 3.33).

Each of these effects and the likelihood of occurrence are discussed in Sections 3.7.7.1 and 3.7.7.2.

3.7.7.1 *Category O carbonate rocks*

Dense, old carbonate rocks in the strong to very strong range have been crushed and processed to provide coarse and fine filter materials for many existing dams. The authors have found no report of a dam incident or failure due to malfunction of any of these filters. However, not all dams have instruments sufficient to monitor the performance of all of their filters. Also the critical filters (Figure 9.2) may be seriously tested only in an emergency, e.g. when a crack develops in a dam core or the foundation beneath either of the dam shoulders is disrupted.

(a) *Change of grading due to dissolution.* The critical filter zones in most large dams are often less than 2.8 m wide. According to Table 3.6, if pure water was to flow at a rate of 3×10^{-4} m/s through such a zone formed by 50 mm diameter calcite particles, it would lie entirely within the "solution zone". That is, the water would never become saturated with carbonate and the particles would be

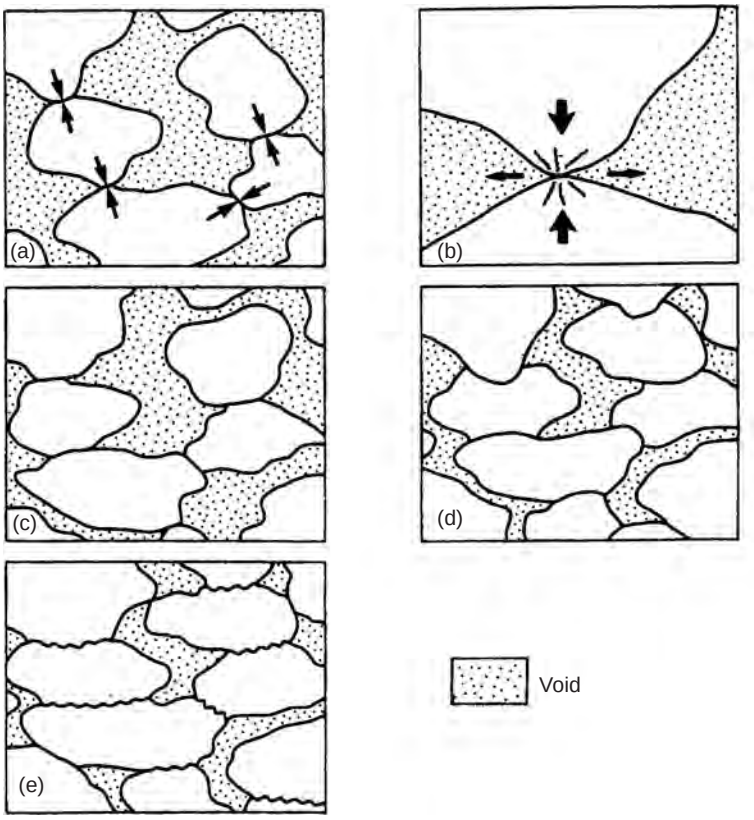


Figure 3.33 Progressive effects of pressure solution in granular materials (Harwood, 1988). (a) Point grain to grain contacts (arrowed), (b) Stressed grain to grain contact (large arrows) leading to formation of dislocations in crystal lattice and subsequent dissolution, with lateral fluid transport of solutes (small arrows), (c) Planar grain to grain contacts, (d) Interpenetrating grain to grain contacts, (e) Sutural grain to grain contacts.

progressively reduced in size, by dissolution. With much smaller particles, as in a fine filter zone, with acidic water and an aggressive reaction the size reduction might become significant during the life of a dam.

(b) *Partial dissolution and recementation.* There have been many accounts of recementation of crushed carbonate materials, due to acids formed by the oxidation of sulphide minerals (See Section 2.9.4). Several examples have been in South Australia where cemented zones have occurred in pavements, embankments and stockpiles formed by crushed rocks of Cambrian and Precambrian Ages. Tests have shown the cementing materials to be calcium sulphate (gypsum) in crushed marble and magnesium sulphate in crushed dolomites. Both the marble and dolomite have performed well as base courses in pavements.

The minus 30 mm crushed marble is well graded and contains 5% to 10% fines (all calcite) with plasticity index of 2. Despite Los Angeles Abrasion Losses of

60% or more, its performance in pavements is remarkably good. Falling Weight Deflectometer tests on a sealed pavement built in 1998 showed its initial modulus to be 700 MPa. The same test after 4.6 years of service showed 1700 MPa. There was no rutting, permanent deflection or change in density (Andrews 2003). The authors consider that the formation and deposition of the gypsum cement, and possibly some associated interlocking of the marble particles due to pressure solution (Figure 3.33) have been the main causes of this increase in stiffness with time.

(c) *Interlocking of grains due to pressure solution.* Both gypsum and magnesium sulphate are highly soluble in water. This raises the following question. Assume that a chimney zone filter has become cemented by either of these during a period of very low and zero seepage. If now increased flow occurs through a crack in the core and cemented filter, how would this filter behave as its cement becomes re-dissolved? The results of testing by Hazell (2003) suggest that the filter might retain the crack or "hang up". The test used was developed to assess the effects of chemical stabilising agents in base materials for unsealed roads. Samples are prepared by removal of the plus 2.36 mm fractions and compacting the remainder at OMC to 98% MMDD in 100 mm long by 100 mm diameter moulds. After removal from the mould samples are bench dried for 7 days. Each is then loaded by an annular weight of 1560 g, through which water drips on to it at a rate of about 400 ml/h. Hazell (2003) advises that during this test all other local non-cementing road base materials (most are crushed quartzites) collapsed within 24 hours, but the crushed marble remained intact, although completely saturated and soft to touch, for at least 460 hours. Assuming that by this stage the gypsum cement would have been completely dissolved, then the sample must have gained strength from some other process. The authors suspect that particle interlock may have developed due to pressure-solution at the grain contacts.

The process and effects of pressure solution in quartz sands in geological time frames are described by Dusseault and Morgenstern (1979), Palmer and Barton (1987) and Schmertmann (1991). Harwood (1988) discusses the role of this process in the reduction of intergranular porosity of sands. Pressure solution effects should proceed much more rapidly in carbonate sands than in quartz sands, because the solution rates of the carbonate minerals are much higher than that of quartz. Relatively inexpensive research (e.g. examination of thin sections cut through epoxy-impregnated old pavements) may show whether or not the effects are rapid enough to render filters composed of *Category O* carbonates ineffective.

3.7.7.2 *Category Y carbonate materials*

(a) *Change of grading due to dissolution.* Sedimentary geologists consider that in fresh water high magnesian calcite is more soluble than calcite and aragonite (Prothero and Schwab 1996). Quantitative data on the solubility and solution rate of high magnesian calcite are not presently available. However, from the field and laboratory evidence discussed below, the authors believe that *Category Y* carbonates used as filter materials would suffer more rapid dissolution than equivalent *Category O* carbonates.

(b) Partial dissolution and recementation. Netterberg (1975) discussed very high CBRs yielded by pedogenic calcretes in Southern Africa and stated "It is difficult to ascribe soaked CBRs much above those yielded by crusher-runs to anything other than cementation". He also included test results showing that the CBRs of some calcretes had more than doubled after a few cycles of wetting and drying. In Netterberg (1969) he described studies aimed at understanding how these calcretes were formed in soil profiles. He concluded that fine carbonate material in the soil is dissolved and redeposited as cement during changes in soil water suction and partial pressure of carbon dioxide. He also provided estimated rates of calcrete formation in nature, ranging from 0.02 mm/year to 1 mm/year (Netterberg 1969, 1978). In Netterberg (1971) he suggested that the apparent "self stabilisation" of calcretes used in pavements may be caused by similar (solution and redeposition) processes. The authors have seen many examples of recemented calcrete pavements in Australia.

Stearns (1944) and Tomlinson (1957) describe evidence of recementation of coral sands and gravels, where used as base materials in airfield pavements.

Coquina limestone of Tertiary age is used widely (when crushed) as base course for highways in Florida. This rock comprises corals, shell fragments and quartz grains. Graves *et al.* (1988) report that the strength increase which occurs with time in these pavements is due to dissolution and re-precipitation of fine carbonate particles. They support this claim using results of the LBR (limerock bearing ratio) test, a modified version of the CBR. Quartz sands were mixed in different proportions with carbonate sands containing less than 2% of fines. The samples were compacted into LBR moulds and tested after soaking continuously in water for periods of 2, 7, 14, 30 and 60 days. All samples with 40% or more of carbonate sand showed strength increases ranging from 23% to 65%, after the first 14 days of soaking. No further increase was recorded after this time. Graves *et al.* Suggested that the very fine carbonate particles may have been "used up" by the end of 14 days and that the coarser grains were not providing cementing material as efficiently. Tests with crushed carbonate base course materials containing "slightly more carbonate fines" were tested similarly for 30 days and showed increases in strength up to that time.

McClellan *et al.* (2001) carried out further laboratory research into these effects but were unable to reproduce comparable strengthening of samples within curing times of up to 60 days.

(c) *Interlocking of grains due to pressure solution.* Shoucair (2003) has advised that the Florida Department of Transport has been adding coral sands to their road bases with significant strength gains. However he notes that the gains "practically disappear when the materials are soaked again" and considers that mechanisms described by Schmertmann (1991) are more likely causes of the initial strength gains than solution and recementation of the carbonate material.

One of the mechanisms described by Schmertmann was interlocking of grains due to pressure solution and the authors consider it possible that this may be the cause of the small remaining strength after re-soaking.

Based on their experience with the gypsum-cemented marble (see Section 3.7.7.1) the authors suggested that the initial strength increase might have been caused by the

presence of minute amounts of gypsum and that this could have been present in the coquina or in the waters used. Mcclellan (2003) has advised that the presence of gypsum is possible, even likely, during road construction in Florida. Traces of pyrites occur in some of the limestones and sea sprays (containing sulphates) occur widely. However he believes that gypsum is not a factor in their laboratory tests. SEM/EDX examination of the cemented areas of their specimens has never shown any significant sulphur.

Some very weak, porous calcarenites are sawn into blocks, which are then used for the walls of small buildings. Newly-sawn blocks are soft to touch and easily scratched, gouged or broken. After exposure to the weather (wetting and drying) for several years, the surfaces of the blocks become noticeably stronger and more durable. It appears that this "case-hardening" happens because carbonate derived from dissolution within the blocks becomes deposited near their surfaces. The Gambier Limestone (Tertiary Age) in South Australia is an example. Millard (1993) notes that "soft local limestone" on Malta, shows similar behaviour. Similar "case-hardening" effects can be seen in the faces of many cliffs formed by *Category Y* limestones, and carbonate-cemented sands.

The following conclusions can be drawn about *Category Y* carbonates:

1. When subjected to compaction, followed by soaking or wetting and partial drying, some of them have become strengthened by solution and recementation of carbonate grains in a few days to a few years. In most cases the above has involved some very fine carbonate material. In some cases most of the strength gains have been lost on re-soaking.
2. Exposed surfaces of some very weak, porous carbonate rocks become strengthened in a few years by solution and re-deposition of carbonate.
3. In each of the above cases it is possible that the strengthening has been partly or wholly caused by gypsum.

The moisture environments in filter materials (in chimney zones, in particular) may at times be similar to those in which cementation was postulated, in 1 above. A carbonate filter zone would not normally contain significant amounts of fines. However the authors consider that the possibility exists that during the lifetime of a dam, solution and recementation and/or grain interlock from pressure solution, could render it cohesive and ineffective.

The authors consider that carbonate materials should not be adopted for filters until the possible long-term effects of dissolution have been assessed by geotechnical specialists. These are unlikely to support the use of carbonate materials for fine filters, but may find they are acceptable for coarse filters in some circumstances.

3.7.8 Suitability of carbonate rocks for embankment materials

Category O carbonate rocks have been used widely and with apparent success as rockfill, random fill and riprap. However, there is some potential for deleterious effects, if sulphide minerals are present:

– in the carbonate rocks,
– in other materials in the embankment or
– in the foundation or storage area.

Sulphuric acid formed by oxidation of the sulphides may attack the carbonate rock and form calcium or magnesium sulphates, which could become deposited in adjacent filter zones.

Carbonate rocks commonly occur in association with mudrocks containing sulphide minerals and carbonate rocks themselves often contain sulphide minerals. In Australia sulphide minerals can occur in all rock types and have been found at the sites of many dams. Quite small concentrations (much less than 1%) of these minerals can have deleterious effects.

The authors believe that it is best to avoid using carbonate rocks (of any age) as embankment materials, if sulphide minerals are present in any of the above ways, as the solution products could clog filters. In any rock or at any site, it can be difficult to determine the amount of sulphides present and their distribution.

3.7.9 Suitability of carbonate rocks for concrete and pavement materials

Dense carbonate rocks in the strong to extremely strong range have been used extensively for the production of aggregates for concrete, bituminous concrete and pavements. However, a few dolomitic rocks have been found to react with alkalis in Portland cement causing expansion and cracking effects like those from alkali-silica reaction. Experiences at sites where this has occurred are described by Luke (1963), Highway Research Board (1964) and Huganberg (1987). Guillott (1975, 1986) describes petrographic work on reactive carbonate rocks and concludes that only very fine-grained dolomitic rocks containing some clay are likely to cause expansion.

Some carbonate rocks contain nodules or beds of chert (extremely fine grained or glassy silica). Alkali-silica reaction is likely to occur if these rocks are used as aggregate with high alkali cement.

The authors recommend that all carbonate rocks intended for use in concrete be assessed for reactivity.

3.7.10 Stability of slopes underlain by carbonate rocks

Natural landsliding is not common in areas underlain by pure carbonate rocks. In weathered, solution affected carbonate rocks it is common to find that joints, faults and bedding partings have been partly or wholly "healed" by redeposited calcite. This along with their inherently high frictional strength of joints seems likely to be the reason for the low frequency of landslides.

In the experience of the authors most slides in carbonate rocks have occurred along interbeds of mudstone or shale. Figures 3.34 to 3.36 show a landslide of approximately 2500 million m^3 in folded limestone of Tertiary Age in Papua New Guinea. The slide is believed to have occurred into an abandoned valley of the Mubi River, due to daylighting of a thin mudstone bed within the limestone (Figure 3.36). The slide was probably triggered by earthquakes associated with fault movements, which displaced the Mubi River about 400 m laterally, and by continued uplift and tilting of the limestone to the south of the fault.

The Bairaman landslide, also in Papua New Guinea, occurred in a 200 m thick horizontal bed of limestone (King *et al.*, 1987) and its horizontal basal failure surface

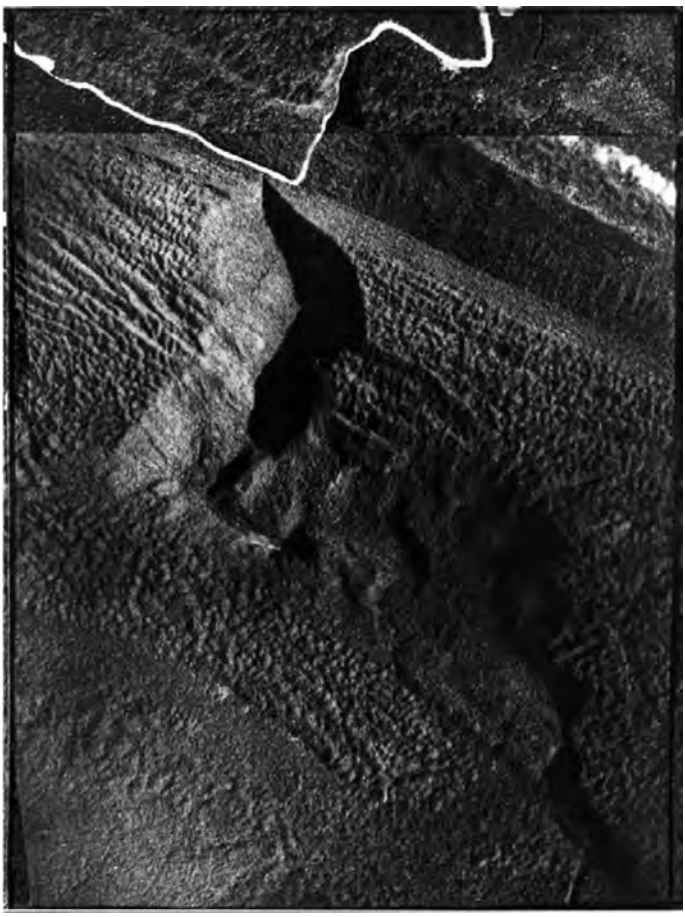


Figure 3.34 Air photo showing the scar of an old landslide in folded limestone, near the Mubi River, Papua New Guinea.

was semicircular in plan, covering about 1 km^2. Its basal surface must have been a mudstone bed.

In detailed studies of carbonate rocks in the Vaiont landslide Hendron and Patton (1985) have shown that clay-rich units and clay are present along and near most of the failure surface.

James (1983) draws attention to situations in Sri Lanka where solution of near-horizontal limestone beds near valley floors has caused collapse, undercutting and landsliding in overlying beds. Retreat of these beds has produced broad valleys bounded by steep escarpments, with hanging tributary valleys. James considers these almost "glacial-like" valleys to be useful indicators of the presence of limestone in areas devoid of outcrop.

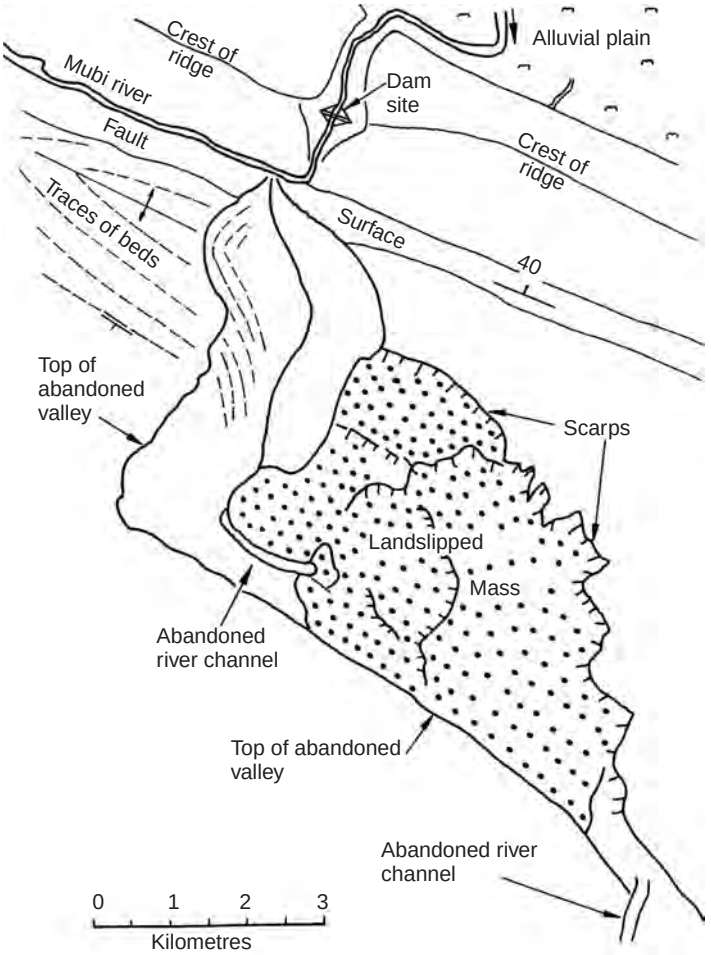


Figure 3.35 Geological plan of the area shown on Figure 3.34.

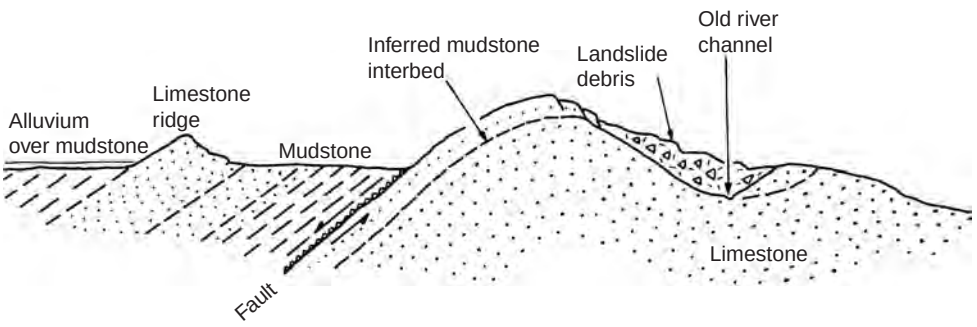


Figure 3.36 Sketch section through the area shown on Figure 3.35.

3.7.11 Dewatering of excavations in carbonate rocks

Excavations below the water table in carbonate rocks containing cavities are likely to require more continuous pumping at higher rates than equivalent excavations in non-soluble rocks. Also because the sizes and distribution of cavities are usually so variable and irregular, a number of borehole pumping tests at carefully selected locations may be needed to get a reliable estimate of the inflow rates.

3.7.12 Carbonate rocks – check list of questions

- Category O or Y?
- Cavities, air-filled or water-filled?
- Cavities, soil-filled?
- Collapse of cavities?
- Extremely irregular, often pinnacled surface of fresh rock?
- Sharp boundary between residual soils and fresh rock?
- Strong rock around solution tubes and cavities in weak, porous rocks?
- Solution cavities in altered carbonate rocks or metamorphosed impure carbonate rocks?
- Very weak, low density, erodible weathered materials?
- Extremely high permeabilities?
- Extreme variations in permeability?
- Possible deep, major leakage paths out of reservoir?
- Presence of sinkholes, exposed or concealed?
- Composition and pH of the groundwater and reservoir water?
- Presence, amounts and distribution of any sulphide minerals?
- Potential for dangerous ongoing solution in the dam foundation?
- Suitability for use for embankment materials?
- Suitability for use in concrete and pavements?
- Alkali-carbonate reaction?
- Chert present: Alkali-silica reaction?
- Shaley (argillaceous) rocks: Durability?
- Unstable slopes, where interbeds of mudrocks are present?

3.8 EVAPORITES

The common evaporites, gypsum ($CaSO_4 \cdot 2H_2O$) and halite (NaCl) are formed in arid areas by evaporation from inland seas, from inland and coastal salt lakes and tidal flats (Stewart, 1963; Murray, 1964). They can occur as individual beds in sedimentary rock sequences, often in association with or interbedded with carbonates. They occur also as matrix, cement, nodules, veins or joint fillings, in mudrocks or sandstones. Anhydrite ($CaSO_4$) is a less common evaporite. Most anhydrite is believed to be formed from gypsum, when it is buried at depths greater than 150 m and its chemically attached water is removed due to overburden pressure and heat. Anhydrite also occurs as nodules or infilling cavities in limestone and dolomite (Murray 1964; Brune, 1965).

The evaporites are much more soluble than the carbonates and only outcrop in arid regions. Sequences containing evaporites show a wide range of solution effects related

to present and/or past groundwater levels. Brune (1965), Bell (1983b, 1993), Hawkins and Pinches (1987), James (1992), Hawkins (1998) and Yilmaz (2001) describe such effects and related ground engineering problems. These include:

- karst topography with extensive systems of caves, tunnels, depressions, chimneys and sinkholes,
- ongoing subsidence, on small and large scales; very slow or sudden collapse,
- leakage through or into solution cavities,
- ground weakening and/or increasing permeability due to ongoing solution,
- heaving ground, due to growth of gypsum crystals,
- explosive heaving/uplifting of ground due to the hydration of anhydrite to become gypsum. Brune (1965) describes how creek channels in Texas were cracked open and uplifted several metres along distances of up to 300 m during such explosions.

3.8.1 Performance of dams built on rocks containing evaporites

The authors know of no dam built at a site containing thick beds of halite. Many dams and reservoirs have been built successfully on sites containing gypsum. A few have later suffered some degree of distress, caused by ongoing dissolution of this material from their foundations. James and Kirkpatrick (1980) refer to Poechos Dam in Peru, founded on clay shales with gypsum, noting that seepage from its right "dyke" was saturated in calcium sulphate and also contained 3.5% of sodium chloride. Notable embankment dam examples in the USA include McMillan and Cavalry Creek Dams (Brune, 1965), Tiber Dam (Jabara and Wagner, 1969). San Fernando Dam (concrete gravity) was affected by rapid solution of gypsum cement from very weak conglomerate (Ransome, 1928; Jansen, 1980).

A more recent example with a gypsum-related issue is Caspe Dam in Spain. Cordova and Franco (1997) cite dissolution of gypsum in the foundation as the cause of a large leakage and damage to this 55 m high embankment dam in 1989 during its second filling. Another is Tarbela Dam in Pakistan, where the right abutment rocks are intensely fractured and include gypsum and "sugary" limestone which is friable and erodible. Grouting has been only partially effective in controlling flows which have high contents of dissolved solids. Additional drainage adits have been required (Amjad Agha, 1980).

Reports on the failure of Quail Creek Dike (James *et al*., 1989; O'Neill and Gourley, 1991) provide useful details on the weathered profile developed on the interbedded dolomite, siltstone and "silty gypsum" which formed the foundation of that 24 m high embankment dam. Its failure was not related to any post-construction solution effects, but was due largely to the failure during design to recognize and take into account the gaping joints in the near-surface rock produced by the past solution of gypsum.

3.8.2 Guidelines for dam construction at sites which contain evaporites

Predictive models derived by James and Lupton (1978) for the rates of dissolution of gypsum and anhydrite have been discussed briefly above in Section 3.7.4. James and Lupton also provide guidelines for the investigation of dam or reservoir sites where

these minerals are present and indicate the kinds and levels of risks associated with construction and operation at such sites. They point out that it is important to know the chemical compositions of the waters involved, because the solution rates of both gypsum and anhydrite are increased by the presence of sodium chloride, carbonate and carbon dioxide. They note that conglomerates which are cemented by gypsum or anhydrite "can produce a material which is potentially very dangerous" if small proportions of the cement are removed by solution. They conclude that the risks of accelerating solution effects are higher with anhydrite than with gypsum and that an "efficient cutoff" is the only practical method to reduce seepage velocities to values low enough to provide "complete safety" against solution effects in "massive anhydrite". They also warn about the possibility of anhydrite converting to gypsum and expanding, with the potential for heave.

James and Kirkpatrick (1980) confirm the conclusions of James and Lupton and provide further predictive data for gypsum and anhydrite and comparable data for halite (see Tables 3.5 and 3.6). They stress the need for chemical tests on drilling water and cores, when halite is suspected, and special sampling procedures if halite is found. They conclude "It would be most unwise to build a dam on massive halite. Unconsolidated strata containing sodium chloride may be unavoidable and control measures are feasible, if costly".

The authors acknowledge the valuable work carried out by James and Lupton (1978), James and Kirkpatrick (1980) and James (1992) and endorse their views, subject to the following:

- We would recommend extreme caution to those considering dam construction at sites where thick beds of anhydrite occur at depths shallower than 150 m.
- As for sites on carbonate rocks (see Section 3.7.6) we would have some reservations about the effectiveness of cement grouting at sites containing thick beds of evaporites severely affected by past weathering and solution. Special attention would be needed to seepage monitoring and facilities for re-grouting during operation of the dam.
- The possibility that filter zones could become cemented with gypsum needs to be considered.
- It is our view that engineers intending to build at sites containing evaporites would be wise to get advice from persons with special knowledge of the evaporite minerals and from others with past experience of construction at such sites.

3.8.3 Evaporites – checklist of questions

- Cavities, air-filled or water-filled?
- Cavities, soil-filled?
- Collapse of cavities – subsidence?
- Ground weakening due to ongoing solution?
- Increasing permeability due to ongoing solution?
- Heave due to growth of gypsum crystals?
- Large scale heave due to hydration of anhydrite?
- Chemical composition of groundwaters/reservoir waters?
- Presence of halite – chemical tests?
- Possibility of cementation of filter materials by gypsum?

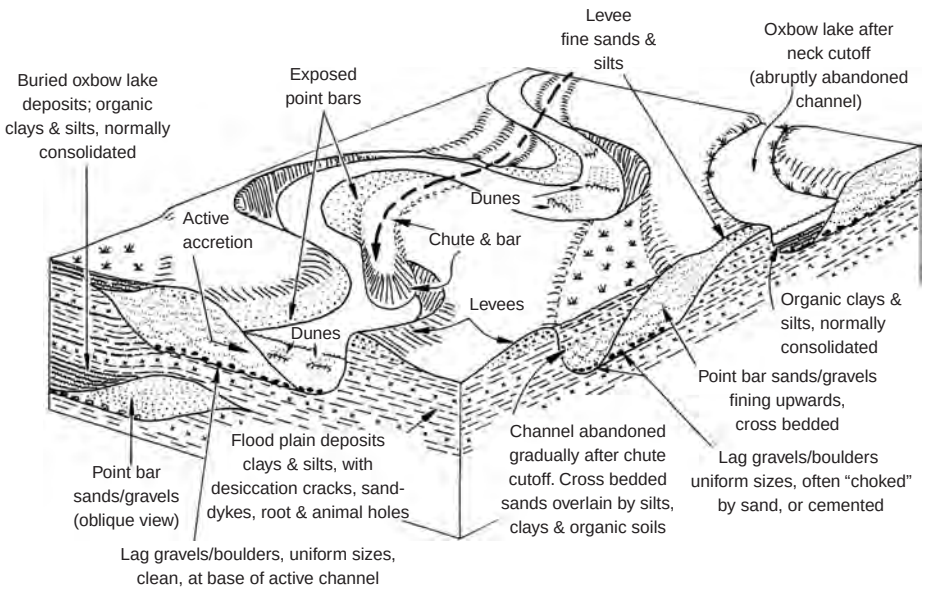


Figure 3.37 Schematic view of soils deposited by a meandering river (based partly on Figure 1 of Walker, 1984).

3.9 ALLUVIAL SOILS

For the purposes of this discussion "alluvial soils" includes soils which have been deposited in the channels and flood-plains of rivers and in lakes, estuaries and deltas. These soils are characterised by great variability, both vertically and laterally and can range from clays of high plasticity through to coarse sands, gravels and boulders.

Detailed sedimentological studies in recent years have provided one or more "facies models" for the sediments (soils) deposited in each of the above environments. A facies model includes block diagrams which summarise the compositions and configuration of the bodies of soil deposited in a particular environment, together with an indication of the processes by which they were formed. Figure 3.37 is a block diagram showing key features of the "meandering river" model, which will be the only one discussed in detail here. Further information about meandering rivers and details of the deposits formed in other environments can be found in Leopold *et al.* (1964), Leeder (1982), Selley (1982), Lewis (1984) and Walker (1984).

The main types of deposits shown on Figure 3.37 are as follows:

– Lag deposits, usually gravel or boulders, occur along the base of the river channel and are moved only during peak flood times. They are usually uniform sized and in the active river channel may have very high voids ratio and permeability. Where preserved at the bases of abandoned or buried channels their voids may be "choked" by sand or by fines.

- Point bar deposits, usually sands and gravels, are deposited on the insides of bends in the stream. During normal flows these materials occur above the lag deposits along the whole of the channel with their upper surfaces in the form of migrating dunes. The cross-bedding seen in the bar deposits results from preservation of some of these dunes. The point bar deposits are usually coarser at depth, becoming finer towards the top.
- Levees of fine sands and silts are formed along the top of the river banks where these coarser materials are deposited more quickly than the fine silts and clays when flood-waters overtop the banks.
- Flood-plain deposits are usually fine silts and clays, deposited in thin horizontal layers during floods. In situations where the soils dry out between floods, desiccation cracks are formed and these may be preserved as sand infilled joints if wind-blown or water-borne sand is deposited in them. Small tubular holes left by burrowing animals or decomposing vegetation are also common in floodplain deposits. These may be preserved open or else infilled by sand or fines.
- Oxbow lake deposits occur in lakes formed during floods when parts of the meandering channel are cut off from the stream. The nature of the deposits depends on whether the channel is abandoned slowly (chute cutoff) or abruptly (neck cutoff) as shown. In each case the fine-grained soils are usually near normally consolidated and are at least partly organic due to the presence of rotted vegetation.

Not shown on Figure 3.37 are bar deposits which can be seen at low flows along straight parts of meandering streams and also in the channels of "braided" streams. Such bars may migrate quite rapidly with aerial photographs taken 10 years apart showing significant changes in the stream bed geometry.

Cary (1950) reports that gravel bars in several fast-flowing rivers in U.S.A. contain elongated lenticular deposits of essentially uniform-sized "open-work" gravel. He reports many open-work gravel lenses in glaciofluvial deposits in the north-western U.S.A. and comments on the large voids and extremely high permeabilities of these materials. The authors have seen open-work gravels and cobbles in which the large voids have become "choked" with sand, which clearly must have migrated into the voids after the original deposition of the gravels.

Cary considers that open work gravels are probably formed in fast-flowing rivers at the downstream ends of rapidly aggrading bars. He suggests that in these situations eddies sometimes occur, which remove finer gravels and sand, leaving only the coarse materials which form the open-work deposits.

In arid or semi-arid climates where stream flows are intermittent, it is common to find cemented layers in the lag and lowest bar deposits of meandering steams and of other, e.g. braided streams. This cementation occurs as the waters dry up and the most common cements include gypsum, calcite and limonite. Cementation is relatively common also in the floodplain deposits.

It is not uncommon to find timber, in some cases the remains of large trees, buried in channel or floodplain deposits. The timber is often well preserved, particularly where groundwaters are highly saline. In some situations the timber is partly or wholly rotten and may have left gaping voids in the alluvial deposit.

It is difficult to generalise about the properties of alluvial soils, because of the extremely wide range of soil types. The following are some observations which may be taken as a general guide.

3.9.1 River channel deposits

The sands, gravels, cobbles and boulders are often highly permeable, particularly in the horizontal direction. Layers, often thin, of finer or coarser materials cause marked differences between vertical and horizontal permeability. This is shown diagrammatically in Figures 3.37 and 10.17. Clean gravels can be interlayered with sands or sandy gravels, giving overall horizontal permeabilities 10 times to 1000 times the overall vertical permeability. The relative density of such deposits is variable, but the upper few metres which are most affected by scour and redeposition during flooding, are likely to be loose to medium dense and, hence, will be relatively compressible and have effective friction angles in the range of 28° to 35°. Deeper deposits are more likely to be dense, less compressible and have a higher effective friction angle.

3.9.2 Open-work gravels

At the 143 m high, 2740 m long Tarbela Dam in Pakistan extensive deposits of open-work gravels occur in the 190 m deep alluvium which forms the foundation for the embankment. The alluvium comprises sands, open-work gravels and boulders and boulder gravels in which the voids are sand-filled (i.e. extremely gap-graded materials). The design allowed for underseepage to be controlled by an impervious blanket which extended 1500 m upstream from the impervious core. The blanket ranged in thickness from 13 m near the upstream toe of the embankment to 1.5 m at its upstream extremity. For several years after first filling (1974) many "sink-holes" or graben-like craters and depressions appeared in the blanket, apparently due to local zones of cavitation within the underlying alluvium. Some of the sinkholes were repaired in the dry and others by dumping new blanket material over them through water using bottom-dump barges. The local collapse zones which caused the sinkholes are believed to have formed when excessively high flow rates through open-work gravels caused adjacent sandy layers to migrate into their large voids.

3.9.3 Oxbow lake deposits

Where clays, silts and organic soils deposited in oxbow lakes have not dried out they are near normally consolidated and may be highly compressible. McAlexander and Engemoen (1985) describe the occurrences of extensive oxbow lake deposits up to 5 m thick in the foundation of the 29 m high Calamus Dam in Nebraska, USA. These deposits comprised fibrous peat, organic silty sands and clays and were highly variable in thickness and lateral extent. Testing showed that the peat was highly compressible. Because of concern about differential settlements and cracking in the embankment, the organic materials were removed from beneath the impervious core and from beneath extensive parts of the shoulders.

3.9.4 Flood plain, lacustrine and estuarine deposits

The clays and silts in these deposits are likely to show pronounced horizontal stratification, with each flood or period of deposition resulting in an initially relatively coarse layer fining upwards as the flood recedes. This may result in marked anisotropy in permeability, with the horizontal permeability being 10 times or even 100 or 1000 times the vertical permeability.

The permeability of these deposits is often increased by the desiccation cracks, sand filled cracks, fissures and holes left by burrowing animals and rotted vegetation. Where such defects have been backfilled by clay soils, the permeability of the mass can be decreased.

Where desiccated the clay soils are overconsolidated and their shear strengths are affected by the presence of fissures which are often slickensided.

In environments where the water table has remained near the surface, e.g. coastal estuaries, the flood-plain soils may be very soft clays, which are near normally consolidated except for an overconsolidated upper 0.5–2 m. In these cases the soils at depth may have a very low undrained shear strength and very high compressibility.

3.9.5 Use of alluvial soils for construction

The filters and concrete aggregate for many dams are obtained from alluvial sand and gravel deposits. In many cases the strict grading requirements and need for low silt and clay content (usually less than 5% or 2% passing 0.075 mm) necessitate washing, screening and regrading. The source of the sediments will have a marked effect on their durability. For example, sand and gravel in a stream fed from areas partly underlain by siltstone is likely to have gravel size particles which will break up readily, rendering the gravel unsuitable for filters or concrete aggregates, whereas those originating from areas underlain by granite, quartzite or other durable rocks are more likely to be suitable.

Alluvial clays, sandy clays and clayey sands (including a proportion of gravel in some cases) can be suitable for earthfill zones in dam construction. Because of their likely variability, the deposits need careful investigation to delineate suitable areas. Borrowing with a shovel and truck operation is sometimes necessary to ensure adequate mixing. Some Australian dams in which alluvial materials have been used for earthfill core zones include Blue Rock, Cairn Curran, Buffalo, Eildon (Victoria), Blowering (N.S.W.), Bjelke-Peterson and Proserpine (Queensland), Hume (NSW – Victoria).

3.9.6 Alluvial soils, list of questions

- Vertical and lateral variability related to deposition conditions?
- Lenticular deposits of open-work gravels with extremely high permeability?
- Anisotropy due to layering?
- High kH: kV ratio?
- Oxbow lake deposits, compressible organic soils?
- Cracks, fissures, holes after rotting vegetation or burrowing animals, all either open or backfilled?
- Cemented layers?
- Buried timber, rotten or preserved, large voids?

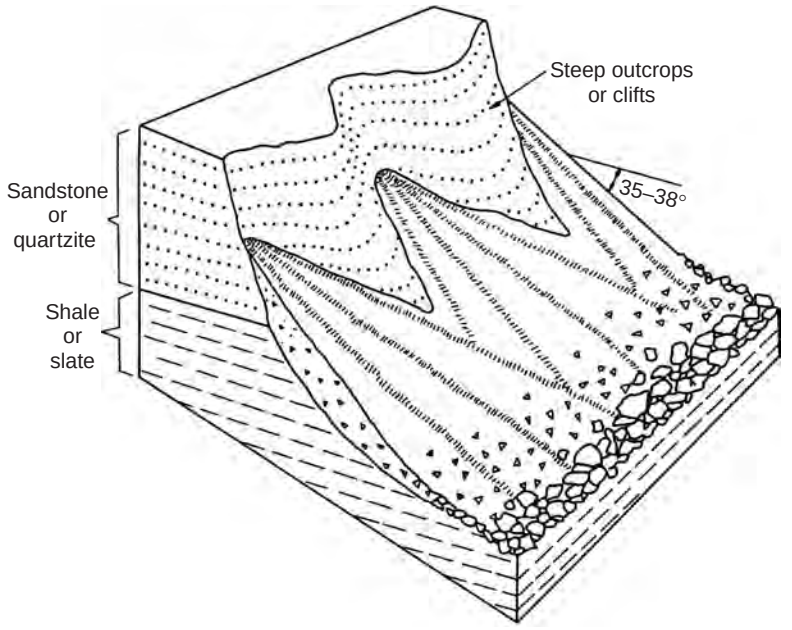


Figure 3.38 Schematic view of scree deposits.

3.10 COLLUVIAL SOILS

3.10.1 Occurrence and description

under this heading are all soils which have been eroded and deposited under gravity forces, often with the aid of water flow. They include slopewash, scree (talus), and landslide debris. The soils range from high plasticity clays through to boulder talus deposits, but are characterised by being mixtures of particles of contrasting sizes e.g. clays with embedded gravel and boulders in landslide colluvium and clayey gravelly sand slopewash deposits. They are also commonly variable within each deposit. Figures 3.38 and 3.39 show typical environments in which scree and slopewash deposits are formed.

3.10.1.1 *Scree and talus*

These are deposits of rock fragments which detach from cliffs or areas of steep outcrops and fall by gravity and roll/slide downslope. The upper scree slopes are composed of smaller rock fragments and usually are at slopes of 35° to 38°; the toe of the slope usually comprises large blocks at flatter angles. The deposits are not water-sorted. They are usually very loose (low bulk density) and just stable at the natural angles. When the deposits contain 30% or more of fine-grained soil, they are called talus.

Selby (1982, 1993) provides a more detailed discussion on the variety of processes of formation of scree and talus and on the range of fabrics resulting from this. Some talus deposits show poorly developed soil profiles near the surface or at intervals

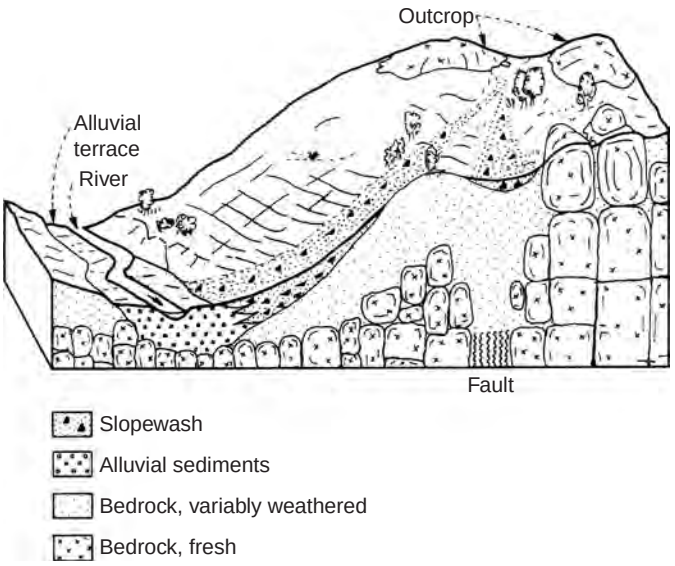


Figure 3.39 Schematic view of residual, slopewash and alluvial deposits.

at depth. These indicate periods during which weathering of the deposit has been proceeding at a faster rate than accumulation of new rock fragments.

It is not uncommon to find timber embedded in scree, either rotted or in a preserved condition.

3.10.1.2 *Slopewash soils*

Slopewash soils are admixtures of clay, sand and gravel which have been moved downslope by the combined actions of soil creep (due to gravity forces) and erosion by water. The thickest deposits are developed in depressions or gullies as shown on Figure 3.39. Near the base of steep slopes slopewash soils often overlie or are intertongued with alluvial deposits (also shown on Figure 3.39).

In cold climates (see Section 3.12) freezing and thawing of the ground can be a major contributing factor in soil creep and deep slopewash deposits are common.

Slopewash soils sometimes show indistinct bedding parallel or non parallel to the ground surface. Slopewash usually has low density and often exhibits tubular voids left by rotted vegetation or roots or burrowing animals or caused by erosion of fines from within the deposit.

3.10.1.3 *Landslide debris*

Landslide debris can range from high plasticity clay through to silty sand from ash flows or sand/gravel/boulder soils resulting from avalanches. In most cases the soils are very variable, vertically and laterally, and it is not uncommon to find large boulders embedded in a clay matrix.

Timber (the remains of trees) is often present in modern landslide debris. It may be well preserved or rotting. Voids left by rotted timber are sometimes found.

Open cracks and irregular voids are often found in landslide debris, particularly where the debris has resulted from or has been affected by modern slope movements.

At sites where landslides have dammed and diverted pre-existing rivers it is common to find landslide debris overlying river alluvium.

Deposits of landslide debris are often underlain by a sheared or slickensided zone (the slide surface) and there may be several sliding and shear surfaces at other levels within the debris. In many cases the main slide surface may be in a zone of material which appears to be residual soil or extremely weathered rock and is characterised by a higher clay content than that of most of the debris.

High groundwater tables are common in landslide debris, but this is not always the case.

3.10.2 Properties of colluvial soils

As for alluvial soils, it is difficult to generalise because of the extremely wide range of soil types. Some general characteristics which may be present are:

3.10.2.1 *Scree and talus*

These materials are likely to be highly permeable, and compressible. As they are sorted they are likely to be poorly graded.

As these materials occur close to their natural angle of repose, excavation into scree or talus slopes usually causes ravelling failures extending upslope. Entry of excessive water (e.g. by discharge from roads) into talus materials can cause them to develop into debris-flows.

3.10.2.2 *Slopewash*

These soils may be more permeable than expected from their soil classification, reflecting the presence of voids and loose structure. They are also likely to be relatively compressible. Many slopewash soils are highly erodible.

Where they occur on steep slopes (e.g. as in Figure 3.39) slopewash deposits are often only just stable. Construction activities which cause such deposits to be oversteepened or to take up excessive amounts of water can result in landsliding.

3.10.2.3 *Landslide debris*

Many landslide debris soils have relatively low permeabilities, but their mass permeabilities may be high, due to the presence of cracks resulting from sliding movements. The shear strength of the colluvium is often reasonably high, but slide surfaces at the base and within the colluvium will be at or near residual. Almost invariably, the soil at the base of the slide is not the same as the slide debris, so shear strength tests on the slide debris can be misleading and usually overestimate the strength. Where the colluvium is derived from fine grained rocks such as shale, siltstone or claystone, the weathered rock underlying the colluvium often has higher permeability than the colluvium, an important point when considering drainage to reduce pore pressures and improve stability.

Landslide debris deposits are often only marginally stable and slope instability may be initiated by minor changes to the surface topography or to groundwater conditions (see Section 2.10.1).

3.10.3 Use as construction materials

Landslide debris and slopewash were used for the impervious core of the 161 m high Talbingo Dam, see Hunter (1982) and Section 2.10.3. The landslide debris, composed mainly of extremely weathered andesite, was used in most of the core. The slopewash, higher plasticity material derived from the landslip deposit, was used as core-abutment contact material.

Slopewash derived from extremely weathered granitic rocks was used for core-abutment contact material in the following dams, for which the parent extremely weathered granite formed the remainder of the core.

- Eucumbene (N.S.W., Australia),
- Dartmouth (Victoria, Australia),
- Thomson (Victoria, Australia),
- Trengganu (Malaysia).

Slopewash derived from extremely weathered rhyodacite was used for the core at Tuggeranong Dam (A.C.T., Australia).

When considering the possible use of landslide debris as earthfill, the critical issues are the potential variability of the soil and the possible need to remove large boulders and cobbles. Also the possibility of renewed slope movements must be considered, where the deposits occur on sloping ground. High groundwater levels and resulting wet conditions may create further difficulties.

3.10.4 Colluvial soil – list of questions

(a) Scree and talus:

- High permeability and compressibility?
- Timber debris, rotted or preserved?
- Potential for instability or debris-flow?

(b) Slopewash:

- Tubular voids causing high mass permeability?
- Compressible?
- Erodible?
- Potential for slope instability?

(c) Landslide debris:

- Variability in composition and properties – laterally and vertically?
- Boulders?
- Large voids?
- Gaping or infilled cracks?
- High compressibility?

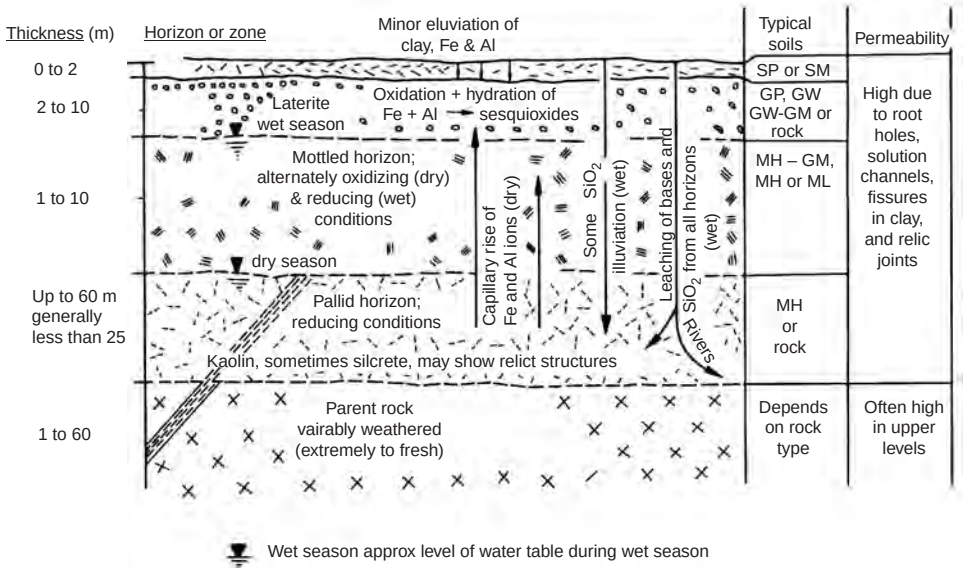

Thickness (m)
Horizon or zone
Minor eluviation of clay, Fe & Al
Typical soils
Permeability
0 to 2
SP or SM
2 to 10
Laterite wet season
Oxidation + hydration of Fe + Al → sesquioxides
GP, GW GW-GM or rock
High due to root holes, solution channels, fissures in clay, and relic joints
1 to 10
Mottled horizon; alternately oxidizing (dry) & reducing (wet) conditions
dry season
Capillary rise of Fe and Al ions (dry)
Some SiO2 illuviation (wet)
Leaching of bases and SiO2 from all horizons (wet)
Rivers
MH – GM, MH or ML
Up to 60 m generally less than 25
Pallid horizon; reducing conditions
Kaolin, sometimes silcrete, may show relict structures
MH or rock
1 to 60
Parent rock vairably weathered (extremely to fresh)
Depends on rock type
Often high in upper levels
Wet season approx level of water table during wet season

Figure 3.40 Diagram showing the lateritic weathering profile of Selby (1982) and some of the processes involved in its development (by permission of Oxford University Press).

- Timber, rotting or preserved?
- High permeability?
- High water tables – wet conditions?
- Old slide surfaces of low strength at the base or at other levels in the deposit?
- Potential for renewed sliding movements?

3.11 LATERITES AND LATERITIC WEATHERING PROFILES

Townsend *et al.* (1982) provide the following definition of laterite:

> *"Laterite refers to varied reddish highly weathered soils that have concentrated oxides of iron and aluminium and may contain quartz and kaolinite. Laterite may have hardened either partially or extensively into pisolitic, gravel like, or rock-like masses; it may have cemented other materials into rock-like aggregates or it may be relatively soft but with the property of self-hardening after exposure".*

3.11.1 Composition, thicknesses and origin of lateritic weathering profiles

The features of lateritic weathering profiles, some of which are shown also on Figure 3.40, are given in Table 3.7 based mainly on Selby (1982).

The "crust" materials are termed ferricrete when they consist mainly of iron oxides. The crust can be either gravel or rock ranging from very weak to very strong, often

Table 3.7 Features of lateritic profile (based on Selby, 1982).

Thickness (m)	*Description*
0 to 2	Soil zone, often sandy and sometimes containing nodules or concretions; this may be eroded away.
2 to 10	Ferricrete or alcrete crust of reddish or brown hardened or slightly hardened material, with vermiform (or vermicular) structures (i.e. having tube-like cavities 20–30 mm in diameter) which may be filled with kaolin; less cemented horizons may be pisolithic (i.e. formed by pea-sized grains of red brown oxides).
1 to 10	Mottled zone; white clayey "kaolinitic" material with patches of yellowish iron and aluminium sesquioxides.
Up to 60 but often <25	Pallid zone; bleached kaolinitic material; the distinction between the mottled and pallid zone is not always apparent and they can be reversed; silicified zone which may be hardened (i.e. silcrete on Figure 3.40).
1 to 60	Weathered rock showing original rock structures.

requiring blasting for its excavation. The very weak rock materials often become stronger when exposed to the weather.

Laterites are believed to have been formed under tropical or sub-tropical monsoonal climates with well defined wet and dry climatic conditions and most laterites of Tertiary Age and younger occur in tropical or sub-tropical areas on both sides of the equator. Lateritic profiles are found also within rocks of Mesozoic and Palaeozoic Ages, mostly at higher latitudes, where their presence indicates ancient tropical conditions which can be explained by continental drift (Bardossy and Aleva, 1990). Laterites have been developed on the full range of common igneous, metamorphic and sedimentary rocks (Gidigasu, 1976).

There are several theories about the formation of laterites (Bardossy and Aleva, 1990; McFarlane, 1976) but most include the influence of a fluctuating water table between wet and dry seasons to allow solution and transfer of soluble silica, iron and aluminium ions, resulting in iron and aluminium oxides accumulating in the upper part of the profile (Figure 3.40).

Lateritic profiles may be much shallower than described above, e.g. in the Ranger Mine area in northern Australia and in many other exposures in Australia and south-east Asia they are less than 5 m thick. Some of these shallow laterite profiles are of detrital origin, i.e. they comprise ferricrete and/or alcrete gravels which have clearly been eroded from an earlier laterite weathering profile and redeposited. These "reworked laterites" show varying degrees of re-cementation and may or may not be underlain by mottled and pallid zones.

Figures 10.18 and 10.19 shows some common features of lateritic profiles in valley situations in northern and western Australia.

3.11.2 Properties of lateritic soils

The most abundant soils in the mottled zone are usually clays, sandy clays or gravelly sandy clays, which behave as soils of medium to high plasticity, but which usually plot below the "A" line in the Casagrande classification chart. Strictly speaking

they are therefore classified as silts according to the Unified Soil Classification. This behaviour is a result of the presence of allophane, kaolin, gibbsite, bauxite and often halloysite.

Other clays have been found in some laterites, for example Gordon (1984) records montmorillonite and illite in profiles developed over dolerite bedrock at Worsley, Western Australia. However this would not be common. Gordon and Smith (1984a, b) describe the results of field and laboratory tests on these and other laterite soils in that area.

As indicated on Figures 3.40 and 10.19 *in situ* laterite profiles are often highly permeable. Many of the structural features which cause the high permeability are near-vertical. In the upper zones vertical tubes, called "channels" or "drains" may have been formed by the decomposition of roots. These may be open or infilled with sand. The mass permeability in these zones may be 10^{-2} m/sec to 10^{-4} m/sec. These near-vertical features are not located readily by conventional drilling and water pressure testing and so in these cases the permeability is best determined by vertical infiltration tests involving relatively large test areas.

3.11.3 Use of lateritic soils for construction

Lateritic soils usually make excellent earthfill construction material. The most notable early case of the use of lateritic soil was in Sasumua Dam, where Terzaghi (1958) showed that despite its apparently peculiar classification properties (i.e. plotting below the "A" line), the lateritic soil was an excellent dam building material. When used as fill, laterite soils are characterised by high effective friction angle and medium to low density and permeability. In most cases they are readily compacted despite often having high and poorly defined water content. For example at Sirinumu Dam, lateritic clays were readily compacted at water contents between 40% and 50%. However, some particularly silty laterites with high halloysite contents can be difficult to compact.

The ferricrete gravels and weak rocks in the near-surface crust zone are used in lateritic areas throughout the world as base or sub-base material in pavements for roads and airstrips. Strongly cemented rock from this zone has also been used successfully as rockfill and riprap.

In the laterite and mottled zone soils are usually non dispersive and in the authors' experience, very resistant to erosion *in situ* and recompacted.

3.11.4 Karstic features developed in laterite terrain

Sinkholes similar to those seen in karstic limestone are known to occur also in some laterite profiles developed on non-carbonate rocks.

Twidale (1987) describes sinkholes up to 50 m diameter and 15 m deep on the Western Stuart Plateau in Northern Territory, Australia. The plateau is underlain by a sequence of siltstones and quartzites, ranging from 40 m to 230 m in thickness, which is underlain by limestone. The laterite profile is developed at the top of the non-carbonate sequence and may be 20–30 m deep. Twidale (1987) suggests that where the sequence is thin some of the sinkholes may have formed by the lateritic materials collapsing

directly into voids in the underlying limestone. However he believes that they most probably developed as follows:

– Voids were formed within the lateritic profile by the removal of silica and silicate minerals by solution, these being carried down joints, deeper into the sequence and
– The ferricrete cap collapsed into the voids.

Twidale believes that the subsurface voids developed in late Tertiary time and that collapse of the ferricrete cap into the voids has continued intermittently since that time. He provides details of the youngest sinkholes, formed in 1982, and describes how many older ones are much modified by sidewall collapse and infilling with slopewash and alluvial soils.

Twidale (1987) refers also to karstic features in laterite developed on peridotite (ultrabasic rock) in New Caledonia. The karstic features in that area occur in broad plateaus surrounded by steep ridges which are remnants of relatively fresh peridotite. They include:

– Internal drainage into sinkholes,
– Recently active sinkholes up to 15 m wide and
– Swamps, believed to represent collapsed areas now filled with sediment and water.

Many of the sinkholes occur in lines above steeply dipping defect zones in the peridotite. The collapses appear to have occurred progressively into voids formed where silica and silicates have been removed by both dissolution and erosion and carried away in near-horizontal and near-vertical flow paths (widened defects) within the defect zones.

The authors suggest that, if such features are evident, investigations should be carried out to establish the mechanism and the potential effect on the dam or its storage.

3.11.5 Recognition and interpretation of silcrete layer

As shown in Figure 3.40, the silcrete formed by deposition of silica in the pallid horizon can contain "relic" structural features, e.g. bedding or foliation which were present in the original rock before it was weathered. It is important that this is understood, to avoid errors in interpretation of drill cores and exposures in excavations. For example, a typical silcrete layer is very strong rock, similar to quartzite and is near horizontal. It is usually underlain by weathered rock with soil or very weak rock properties. If the silcrete was formed within weathered material which was previously vertically bedded rock, it will appear to show vertical bedding. It might then be misinterpreted as part of a vertical layer of quartzite bedrock, continuing to depth.

This type of situation was found during the raising of Hinze earth and rockfill dam in Queensland (see Fell *et al.*, 1992, Page 122). In the right abutment teeply dipping beds of open-jointed and highly permeable chert were interbedded with greenstone and phyllite. Weathering persisted to at least 40 m below the ground surface. It is believed that silica released during lateritic weathering of adjacent metamorphic rocks had progressively replaced carbonate in the uppermost 40 m or so of the greenstone, to form a silcrete horizon. The Stage 3 construction is described further in Section 10.9.1.

3.11.6 Lateritic soils and profiles – list of questions

- Variable laterally and vertically?
- Deeply weathered?
- High *in situ* permeability, vertical and horizontal?
- Low *in situ* density at depth?
- If sinkholes present, their mechanism and effect?
- Fine soils suitable for earth core?
- Gravelly ferricrete or alcrete suitable for pavements?
- Cemented material in crust suitable for rockfill or rip-rap?
- Silcrete horizon or quartzite bed?

3.12 GLACIAL DEPOSITS AND LANDFORMS

During the Pleistocene period, large parts of the earth's surface were covered by sheets of ice, similar to those which occur today in Greenland, Antarctica, parts of Europe and North and South America. The ice moved across the landscape, eroding and reshaping it, and when it melted it deposited the eroded materials. Similar "ice ages" occurred earlier in the earth's history, but with rare exceptions glacial deposits formed at those times have been so modified and strengthened by diagenesis or metamorphism that they are now rocks not greatly different in engineering properties from the surfaces on which they were deposited. The discussion here is therefore limited to effects of the Pleistocene glaciation and of valley glaciers such as still occur commonly in alpine regions (Figure 3.41).



Figure 3.41 Aerial view of the Tasman Glacier in New Zealand. Photo courtesy of Mr. Lloyd Homer, DSIR, New Zealand.

Gaciated landscapes usually have complex histories of erosion and deposition, including for example, the following sequence shown on Figure 3.42.

- Erosion and shaping by rivers (Figure 3.42a),
- Erosion and reshaping by ice, and deposition of glacially derived materials (Figure 3.42b),
- Erosion and reshaping of the new landscape and the glacial deposits, by subsequent rivers (Figure 3.42c).

Such histories have resulted in a wide variety of landscapes and deposits. The deposits vary widely in their engineering properties. Unfortunately, from examination of the mineral content and texture of a soil it is frequently not even possible to determine whether or not it is of glacial origin. If these characteristics are considered in relation to the landforms on or in which the soil occurs it is often possible to confirm a glacial origin and to make more detailed predictions about its history and thus about its likely distribution and engineering properties.

Boulton and Paul (1976) introduced the "land system" form of terrain evaluation as a means of classifying and mapping of sediment sequences and landforms. This land system approach is used by Eyles and Paul (1985) who recognized two main land systems resulting from glaciation.

- Subglacial and supraglacial – characteristic of lowlands where sediments and landforms were formed by large sheets.
- Glaciated valley – characteristic of areas of high relief in which the ice was restricted to valleys.

Other glacially-related environments are:

- Periglacial – in which intense frost action modified the glacially developed landforms and materials.
- Glaciofluvial – in which glacially derived sediments were transported and deposited by water.

The processes and products of each land system and environment are discussed in some detail in Eyles and Paul (1985). Discussion in this present chapter will be limited to the glaciated valley land system (the most significant in dam engineering) plus brief notes on glaciofluvial and periglacial aspects.

3.12.1 Glaciated valleys

Figure 3.43 is a diagrammatic section along a valley glacier during a period of its "advance" down the valley. It can be seen that the ice tends to move over irregularities in its path, behaving in a viscous manner near its base and failing in shear and tension at higher levels. The resulting valley floor is uneven and often contains deep hollows eroded by the ice. Many such hollows are now occupied by lakes, the water is dammed partly by ridges of rock and partly by glacial debris. Streams of meltwater flow beneath the ice and exit at the snout.

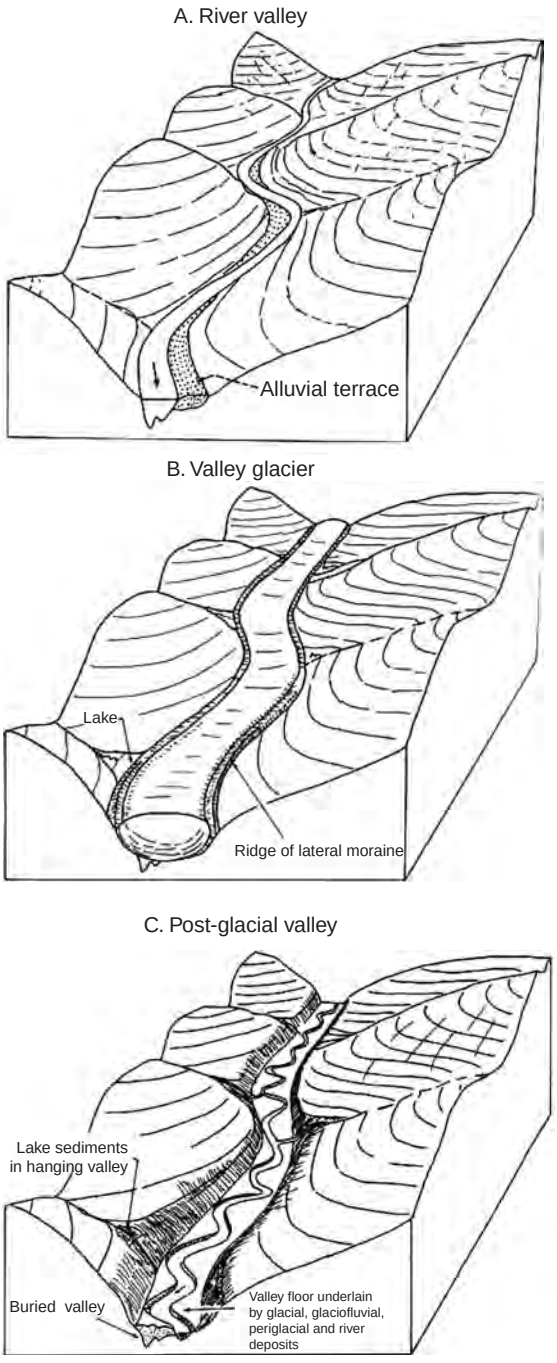


Figure 3.42 River valley, before during and after glaciation.

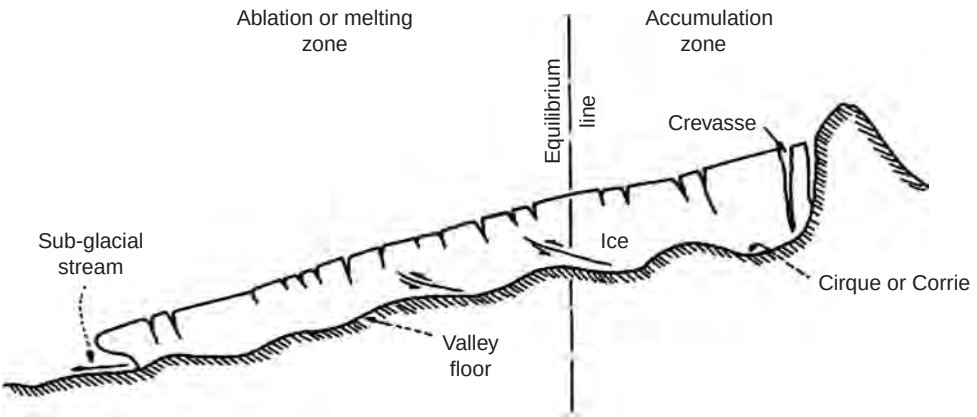


Figure 3.43 Diagrammatic section along a valley glacier (based on Blyth and de Freitas, 1989).

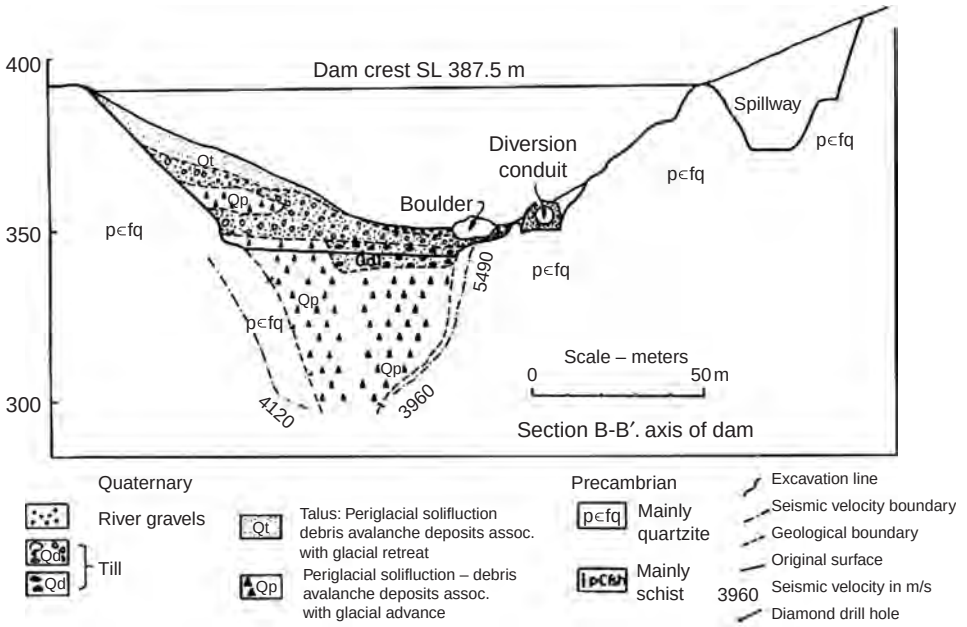


Figure 3.44 Parangana Dam, Tasmania, cross section along dam axis (Paterson, 1971).

During periods in which winter snows do not survive through summer the glacier diminishes in size and its snout "retreats" up the valley.

Although glacial valleys are generally U-shaped when viewed broadly, in detail the valley floors are often highly irregular as at Parangana Dam in Tasmania (Figures 3.44 and 3.45). The shape of the buried valleys at this site and nature of the infill materials indicates that they are remnants of pre-glacial river valleys. Figure 3.44 illustrates the

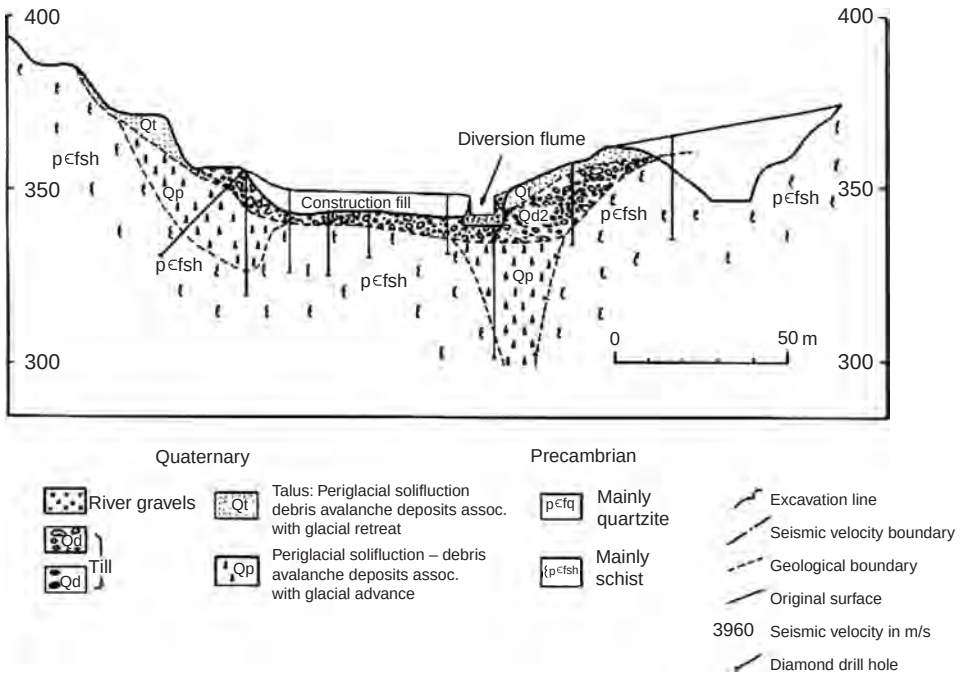


Figure 3.45 Parangana Dam, Tasmania, cross-section 100 m downstream from dam axis (Paterson, 1971).

way in which such a pre-glacial river channel can be preserved under glacially derived deposits.

In other cases locally deeper valley sections under glaciers may have been differentially eroded by ice or meltwater streams.

3.12.2 Materials deposited by glaciers

The general generic term for material deposited by glaciers is "till". Deposits of till are often referred to as "moraine". A further term "drift" has been used extensively and loosely to describe surficial deposits of glacial, alluvial or colluvial origins.

Figure 3.46 shows mechanisms of accumulation, ingestion and transport of debris by a glacier. Upslope from the equilibrium line (Figure 3.46a) windblown dust and rock debris is buried within the snow which feeds the glacier and is transported down glacier forming the basal debris zone and thin basal traction layer. The rock particles in the traction layer abrade, polish and groove the rock floor, generating fine rock particles known as rock flour. The ice and rock blocks within it also pluck or "quarry" rock from the floor and sides. The resulting material developed in contact with the rock floor is known as lodgement till (Figure 3.46b) and usually contains a wide range of particle sizes. The lodgement till is formed under relatively high effective normal pressures and consequently is usually compacted to at least stiff consistency. Its upper surface is grooved or fluted parallel to the direction of flow of the ice.

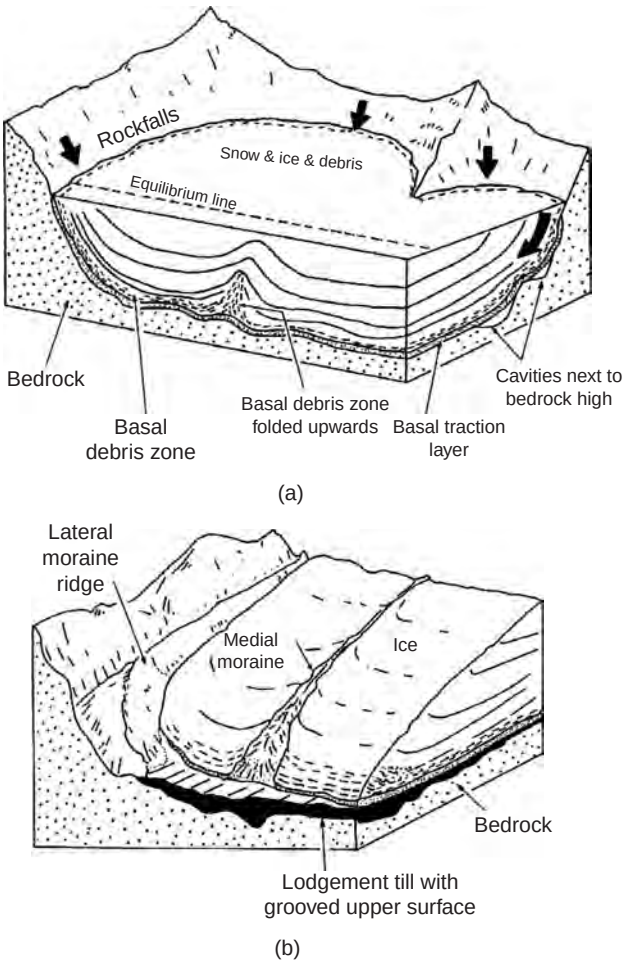


Figure 3.46 Debris transport by a valley glacier (Eyles 1985, by permission of Pergamon Press).

Where two glaciers converge to form a composite valley glacier, the debris zone may be folded upwards by the compression generated along the glacier contacts and in the ablation zone (Figure 3.46b) the debris may become exposed as a supraglacial medial moraine.

Below the equilibrium line (Figure 3.46b) debris falling onto the glacier surface is not ingested by the glacier because the winter snows do not survive the next summer. The debris is transported along the glacier sides as ridges of supraglacial lateral moraine or as supraglacial medial moraine after the convergence of two glaciers.

Figure 3.47 shows a typical situation at the snout where debris from the glacier surface and within it are deposited as the ice melts.

Figures 3.48 and 3.49 show the development of a ridge of supraglacial lateral moraine and soils associated with it during a single advance/retreat stage of a glacier.

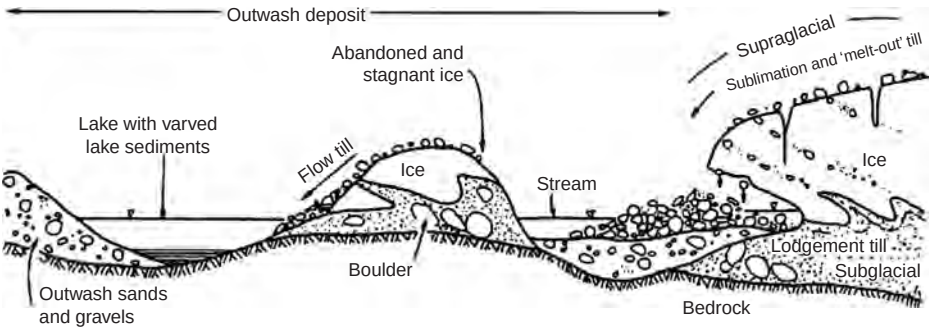


Figure 3.47 Diagrammatic longitudinal section at a glacier snout, and downstream (Blyth and de Freitas, 1989).

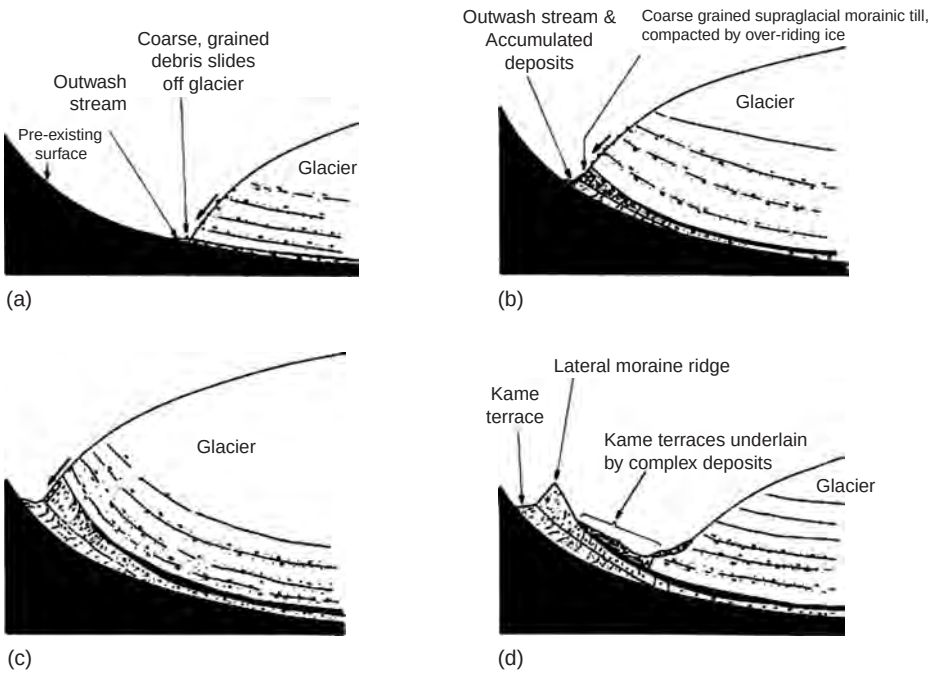


Figure 3.48 Development of a lateral moraine ridge and associated deposits (Boulton and Eyles, 1979).

During the advance (Figures 3.48a to 3.48c), debris which slides off the glacier surface forms steeply dipping deposits (lateral moraine) analogous to talus or end-dumped fill. The material, more precisely known as supraglacial morainic till, includes a wide range of particle sizes. Outwash materials with initial near-horizontal bedding are deposited by streams in the trough between the lateral moraine and the valley side. At depth the accumulated moraine and outwash materials are compressed and compacted by the glacier and their dips become steeper.

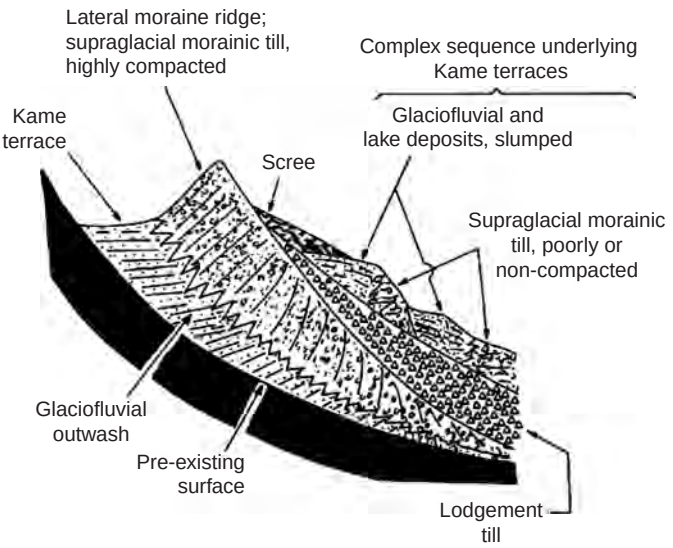


Figure 3.49 Diagrammatic section through a lateral moraine ridge (Boulton and Eyles 1979).

During the retreat (Figure 3.48d) the glacier shrinks away leaving the lateral moraine as a ridge. Such ridges often remain steep (up to 70°) due to the high degree of compaction and some cementation of the moraine. Between the ridge and the ice a series of terraces develops. These are known as kame terraces and are underlain (Figures 3.48d and 3.49) by complex sequences including glaciofluvial sands and gravels (stream-deposits), laminated clays (lake-deposits), poorly- or non-compacted till (supraglacial morainic till) and lodgement till.

Terraces formed by the outwash next to the valley side are also known as kame terraces (Figure 3.49).

The lateral moraine and associated deposits are usually more complex than shown on Figure 3.49 because of repeated glacial advances and retreats. Figure 3.50 shows the relationships between lateral moraine ridges deposited by successive glacial advances. In Diagram (a) Advance 1 was more extensive than Advance 2, and in Diagram (b), Advance 1 was the less extensive.

Figure 3.51 is the complete glaciated valley land system, showing common features of the ablation zone and the deposited materials and landforms. Numbered features on this diagram which need further explanation are discussed below.

The lodgement till (1) at the base of the glacier is poorly sorted, usually containing more than 50% of sand, silt and clay sizes, forming a matrix which supports gravel and larger sized particles. When unweathered, the fines fraction usually comprises finely ground quartz, carbonate and inactive clay minerals. The lodgement till is well compacted by the overlying ice, has a well developed fissured fabric parallel to the ice flow direction and is commonly highly deformed (Figure 3.46). It usually has very low permeability, but may be rendered locally permeable in the mass by thin glaciofluvial sand layers deposited in channels of meltwater streams.

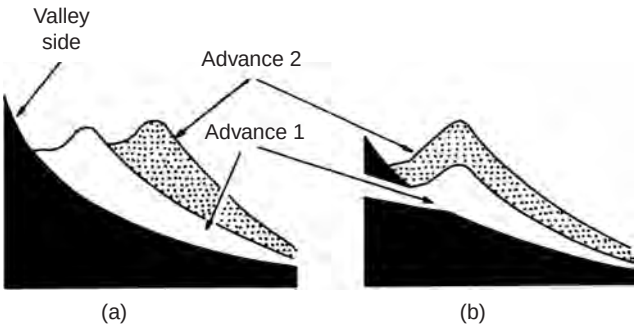


Figure 3.50 Lateral moraine ridges deposited by successive glacial advances (Boulton and Eyles 1979).

The till near the downstream toe at Parangana Dam (Figure 3.45) ranged from "toughly compacted" to weak rock and is considered by Paterson (1971) to be lodgement till.

Drumlins (2) are elongated dune-like mounds on the surface of the lodgement till, apparently moulded to this shape by the moving ice.

Ice-cores (3) are masses of ice left behind by a retreating glacier and buried in till. Melting of ice-cores causes the development of sinkhole-like features known as kettles (4) and deposition of basal melt-out till (5) which is usually crudely stratified.

Flowed till (6) is till which has been reworked and deposited by mudflows, the scars of which can be seen extending from the lateral moraine ridge through its adjacent kame terraces. The flowed till may be stratified due to redistribution of fines during flow. The kame terraces here are underlain by complexly interbedded sediments as shown on Figure 3.49.

Supraglacial morainic till (7) deposited at the snout, as shown on Figure 3.47, contains a large range of gravel and larger fragment sizes and is usually deficient in fines. It is known also as supraglacial melt-out till or ablation till. The younger glacial deposit at the dam axis area at Parangana Dam (Figure 3.44) is believed to be of this type. It comprises up to 70% of gravel to boulder sizes in a matrix of sand, clay and silt. The largest boulders are up to 3 m diameter.

3.12.2.1 *Properties of till materials*

It is clear that basal melt-out till, flowed till and supraglacial morainic till will be poorly consolidated unless they become desiccated or covered by new sediment or ice at a later time in their history. The latter appears to be the case at Parangana Dam where the supraglacial morainic till is well compacted, apparently due to being overridden by ice during subsequent glacier advances (Paterson, 1971). Experience has shown that most tills of all types have low permeabilities (less than 10^{-10} m/s). However, till deposits may contain or be next to bodies of permeable sands and/or gravels of glaciofluvial origin, filling old channels. For example the various outwash deposits shown on Figure 3.48 would normally have relatively high permeabilities. At Parangana Dam the younger till beneath the valley floor contained a layer of glaciofluvial sands several metres thick with measured permeability of 10^{-6} m/s. To allow for

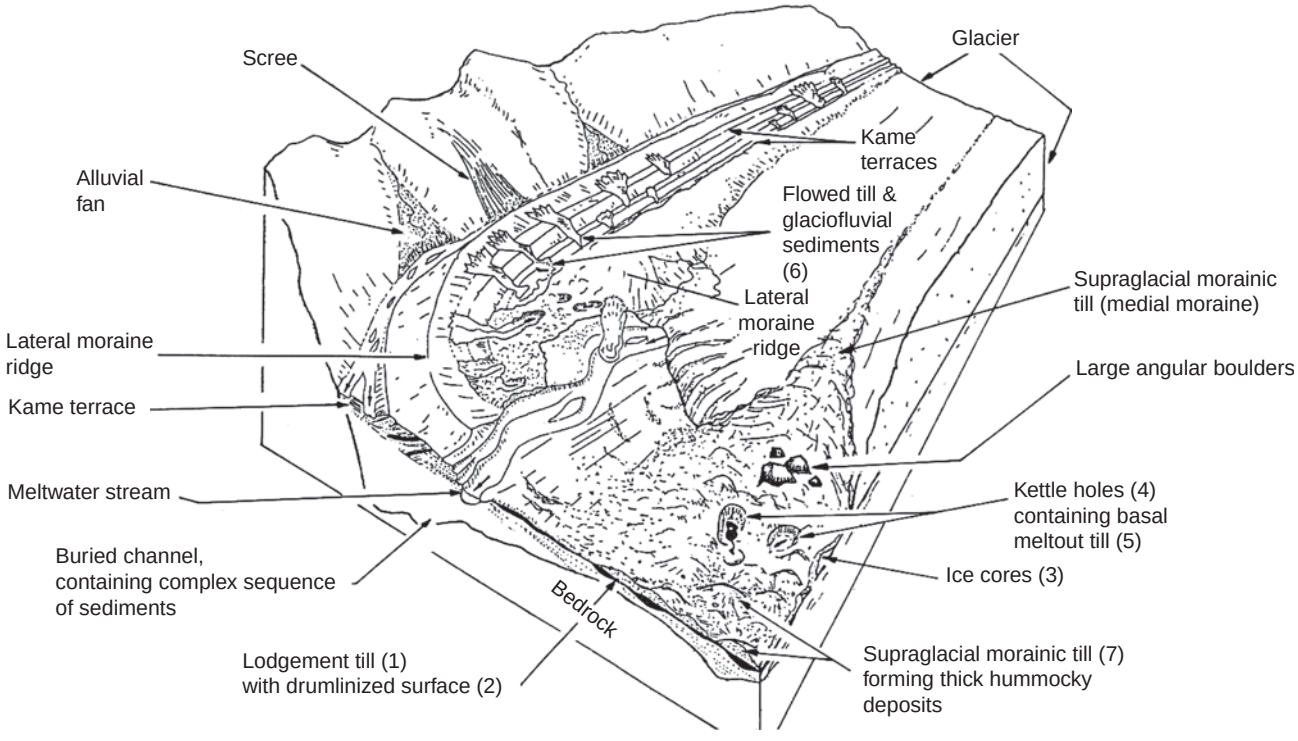


Figure 3.51 Landforms and sediments associated with the retreat of a valley glacier (based on Eyles 1985, by permission of Pergramon Press).

this layer the core cutoff trench was excavated down into underlying materials of low permeability (Figure 3.44). The younger till was left in place beneath the downstream shoulder, but was covered with a filter blanket before placement of the rockfill.

Walberg *et al.* (1985) describe remedial works required at the 23 m high Smithfield Dam in Missouri, USA, after seepages and high piezometric pressures were recorded during first filling. Exploratory drilling using 152 mm diameter cable tool tube sampling showed that glacial outwash sands and gravels beneath the left abutment were more continuous and permeable than assumed from the pre-construction drilling and sampling.

At Cow Green Dam in Britain most of the material filling a buried channel beneath the left side of the valley was found to be lodgement till, described as "*stiff, dark brown, poorly-sorted, unstratified, silty, sandy clay of medium plasticity containing subangular to rounded gravel, cobbles and boulders.*" The boulders ranged up to 2 m in mean diameter (Money, 1985).

Sladen and Wrigley (1985) describe further generalisations which can be made about the geotechnical properties of lodgement tills.

3.12.2.2 *Disrupted bedrock surface beneath glaciers*

Knill (1968) describes the open-fractured nature of bedrock beneath glacial materials at several sites and concludes that gaping or infilled joints near-parallel to the rock surface were initiated as shear fractures by the moving ice (i.e. by "glacitectonic thrusting") and then opened up by ice wedging. The authors have found similarly fractured rock with open and infilled joints at the bases of many valleys which have not been subjected to glaciation (see Chapter 2, Section 2.5.4) and suggest that the effects seen by Knill may have been formed largely by stress relief.

Regardless of their origin, the presence of such features at many sites means that the rock next to the base of glacial deposits is likely to be of poor quality e.g. in terms of compressibility, permeability and erodibility. Where the existence of such poor quality rock would be of significance to the stability and/or watertightness of the dam it is important that this "rock-head" zone be investigated thoroughly.

Money (1985) draws attention to the difficulties often encountered while doing this by core drilling and mentions cases where large boulders have been mistaken for bedrock. He refers to UK practice at that time which was to recommend that the *in situ* rock be proven by a minimum of 3 m of cored rock. He states that this figure is likely to be inadequate and that it is certainly not enough to allow for adequate permeability testing of the upper part of the bedrock. The authors agree and suggest that the actual depth of coring needed will depend upon:

- The inherent fabric of the bedrock (i.e. is it massive or too well-cleaved or closely jointed for it to have formed large boulders?),
- The quality of the core samples and the extent to which core orientation can assist in assessing whether it is *in situ* or not. Core orientation may be determined either by impression packer or orientation device or by the presence of bedding or foliation with known, consistent orientation,
- The actual depth of the disturbed zone.

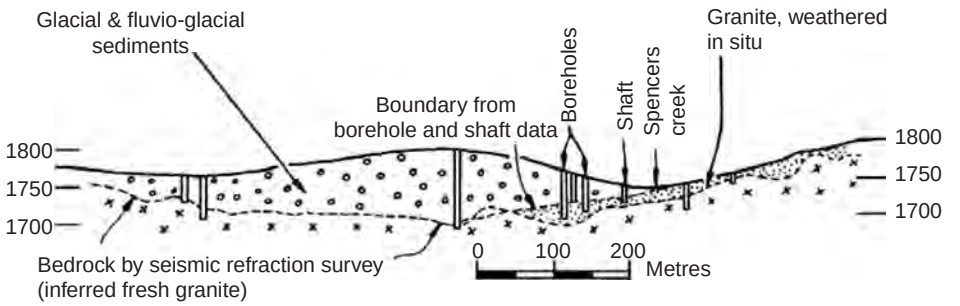


Figure 3.52 Cross section through the site for Kosciusko Dam.

Difficulties in delineation of the top of *in situ* rock can occur also where the rock in this zone has been chemically weathered. This situation exists at the site for Kosciusko Dam in New South Wales (Figure 3.52).

It appears that intense weathering has occurred in both the bedrock and the till. Geological surface mapping, track exposures and boreholes on the right bank show that most of the upper 5–20 m of the bedrock comprises residual "boulders" of fresh to slightly weathered granite set in a matrix of highly to extremely weathered granite which is mainly a very compact silty, clayey sand.

A shaft close to the creek on the left bank (Figure 3.53) showed similarly weathered granite boulders set partly in a matrix of gravelly clay (till) and partly in glaciofluvial sands and clays. It was clear that without very good recovery of little-disturbed core, these materials could not be readily distinguished from the *in situ* weathered "bouldery" sequence. At 8.6 m these materials rested on extremely weathered rock whose mineral content and foliation attitude matched the known bedrock on the right bank. This weathered rock was therefore inferred to be *in situ*.

Holes drilled elsewhere on the left bank recovered about 20% of fresh or slightly weathered granite in "boulder" lengths, but little or no matrix. Hence the upper surface of the *in situ* rock (assuming that it was more than slightly weathered) could not be determined here from the drilling results. The top of mainly fresh granite was inferred from the drill cores together with the results of refraction seismic traverses (Figure 3.52).

Glastonbury (2002), Glastonbury and Fell (2002a,c; 2008; 2010) note that many of the large rapid landslides which have occurred in historic times have occurred in valleys which have been glaciated, and attribute this to the stress relief effects on the valley sides as the glaciers retreat.

3.12.3 Glaciofluvial deposits

As well as depositing some glaciofluvial materials in their immediate vicinity (as discussed in Section 3.12.2) glaciers release very large meltwater flows giving rise to deposition of vast amounts of gravels and sands in braided (multiple-channel) rivers usually extending tens of kilometres downstream, often across broad outwash plains (Figure 3.54). Grain sizes range from gravel-dominated near the glaciers to sands and

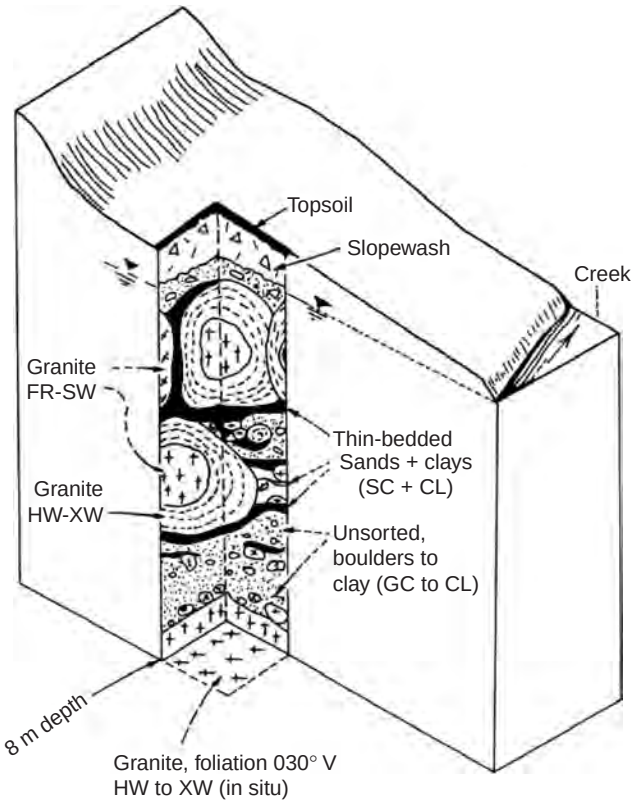


Figure 3.53 Log of shaft, Kosciusko Dam site.

silts further downstream. In some places lakes are formed, in which laminated silts and clays are deposited.

Deposition, eroding and reworking of the braided river deposits occurs cyclically in phase with glacial advances and retreats, resulting in terraces at various levels across the river valleys and outwash plains. The streams eventually flow into lakes or the sea.

Exposed areas of silt and fine sand are eroded by wind and redeposited as loess on the surrounding country.

Miall (1985) describes sedimentological aspects of the braided stream deposits.

The sands and gravels are usually clean with well-rounded particles and often provide excellent sources of materials for embankments and for concrete aggregate.

The braided streams and lakes produce very complex, lenticular deposits of sands, gravels, silts and clays. Terzaghi and Leps (1958) describe the design, construction and performance of Vermilion Dam, a 39 m high zoned earthfill structure built on such complex glaciofluvial deposits, with maximum depth of 82 m.

In some deposits there are lenticular beds of "open-work" gravels or boulders which are uniformly sized materials with large voids and extremely high permeability.



Figure 3.54 Broad valleys downstream from glaciers in New Zealand, underlain by great depths of glaciofluvial gravels and sands. Photo courtesy of Mr. Lloyd Homer, DSIR, New Zealand.

Cary (1950) describes the widespread occurrence of these materials in glaciofluvial deposits in northwestern USA.

3.12.4 Periglacial features

Periglacial conditions are defined here as those under which frost is the predominant weathering agent. They are often, but not always, associated with glaciers. Permafrost conditions are commonly present but are not essential. Permafrost occurs where winter temperatures are rarely above freezing point and summer temperatures are only high enough to thaw the upper metre or so of the ground.

Figure 3.55, modified slightly from a diagram of Eyles and Paul (1985), shows features which may be developed under periglacial conditions:

1. Deep-seated creep of weak sedimentary rocks into the valley. Competent beds develop widely gaping or infilled extension joints (gulls) and move downslope on the weak materials as complex slides or rafts,
2. Weak rocks contorted and bulged upwards, overlain by terraced gravels,
3. Outcrops showing evidence of toppling and cambering, with scree and rockfall deposits downslope,
4. Outcrops of very strong crystalline bedrock surrounded by blockfield of frost-heaved bedrock. Terraces cut by nivation – free-thaw and slopewash at the margins of snow patches,

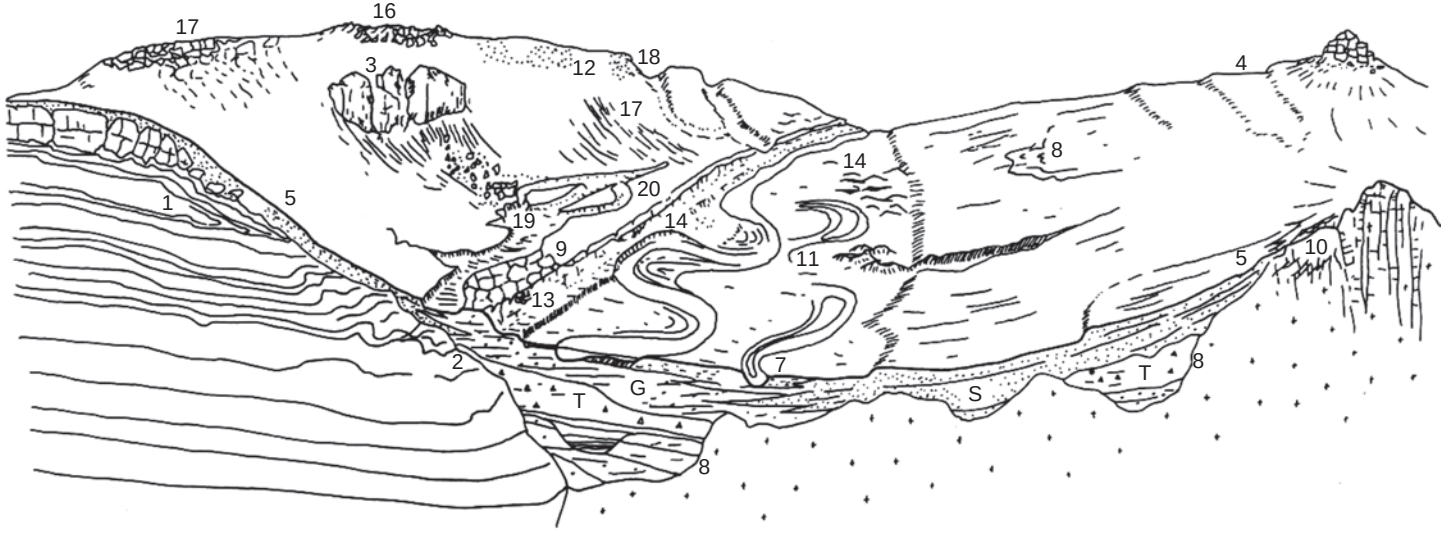


Figure 3.55 Diagram showing features which may be developed under periglacial conditions (Based on Eyles and Paul, 1985).

5. Solifluction fans or lobes – crudely bedded gravelly or bouldery deposits, thickening downslope, grading into slopewash beneath the lower slope,
6. Mudflow scar underlain by low-angle shear surface,
7. Mantle of solifluction debris (S) intertongued with gravels (G) deposited by periglacial braided stream and overlain by peats and silts (M) forming the floodplain of the modern meandering stream,
8. Till (T) in the buried valley and in pockets in the rockhead,
9. Polygonal patterned ground and fossil casts of ice-wedged cracks,
10. Frost-shattered bedrock,
11. Doughnut-shaped degraded ramparts left by former pingos,
12. Soil mantle resulting from mechanical churning (frost heaving) of pedological soils and bedrock. The underlying rockhead is often highly irregular,
13. Involutions and contortions in the soil caused by high pore pressures developed during re-freezing of thaw-soaked soil,
14. Faulted soil zones caused by ground contraction during freezing and/or collapse into voids following melting of buried ice,
15. Patterned ground – polygonal nets and stripes,
16. Blockfield,
17. Alluvial fans,
18. Dry Valleys,
19. Terraced gravels (G) deposited by braided periglacial streams. The bedrock surface below may contain scoured channels and hollows,
20. Loess.

Eyles and Paul (1985) point out that most of the features indicated on this diagram may also develop (more slowly) in warmer climates. The authors agree and in particular consider it likely that the "Deep-seated disturbances" Features 1 and 2 would have been developed to a large extent as a result of destressing (as described in Chapter 2) before any cold-climate effects. The following explanation of the main cold-climate processes and features indicated on Figure 3.55 is based also on Eyles and Paul (1985).

Solifluction (7) is the slow flowage or creep of a water-soaked mass of soil and rock debris, either as a true flow or as a slide where most movement occurs over a basal shear surface. Typical flow rates range from 10 mm to 60 mm per year. Solifluction is caused by the generation of excess pore pressures during the thaw. Eyles and Paul (1985) provide a summary of theoretical studies of the process by Nixon and Morgenstern (1973) and McRoberts and Morgenstern (1974).

Another solifluction mechanism is the down slope displacement of soil particles by needle-ice. The ice needles grow normally to the ground surface and so during each freeze-thaw cycle the supported particles are displaced slightly down slope.

Frost-shatter (10) is the mechanical disruption of rock masses by the expansion on freezing of groundwaters.

Cyroturbation (12), (13) and (15) is the churning or mixing of soils which occurs in permafrost conditions at the end of the melt season due mainly to high pore pressures set up towards the base of the thawed layer as the soil re-freezes from the surface down. Expansion due to freezing of porewaters also contributes, as does the upward heave of cobbles and boulders resulting from their greater thermal conductivity. The "involutions" (13) are pseudo-intrusive structures similar to flame structures in sediments

and to gilgai in clays. Patterned ground (15) is ground showing a regular hummocky pattern (polygons or circles). It is a surface expression of cyroturbation.

Ice-wedging (9) refers to cracks which occur in soils during intense cold widened by wedging action when water freezes in them. The cracks are preserved as casts by soil which migrates into them.

Pingos (11) are conical mounds of buried ice up to 40 m high and 600 m in diameter. Pingos, which have developed by the freezing of upward-moving groundwater, have caused updoming of the surrounding sediments. Melting of the ice results in the doughnut-shaped surface features shown on Figure 3.55.

Paterson (1971) provides a useful account of periglacial and other effects of glaciation features at Parangana Dam. Core-drilling into the steep sided buried pre-glacial river channels at the site (Figures 3.44 and 3.45) showed the lower part of each to be filled mainly by angular rock fragments up to 500 mm across set in a sandy clay to clayey sand matrix. The ratio of rock to matrix was roughly 60:40. The rock fragments were of quartzite and schist, clearly derived from the bedrock immediately upslope in each case. These deposits were judged to have been formed by "solifluction debris avalanches" from the steep valley sides, under periglacial conditions during the glacial advance period.

In the upstream channel the avalanche material contained a discontinuous bed of clay believed to have been deposited in a lake formed upstream from one of the slides. Also present in the slide deposits were lenticular beds of sand and gravel, inferred by Paterson to be stream deposits in channels eroded through the slide dams. The distribution of sand and gravel beds suggested that at least five major landslides had occurred.

Locally derived talus covers most of the ground surface at the site and extends to about 10 m depth in the foundation area (Figures 3.44c and 3.45). This material is believed to be a periglacial solifluction product formed during the final retreat stage of the glacier.

3.12.5 Glacial environment – list of questions

Only the most significant glacier-related questions are listed here; these and other features which are usually of less significance in dam engineering are shown on Figures 3.51 and 3.55.

- Buried valleys?
- Bedrock surface or boulder?
- Bedrock disrupted near upper surface?
- Wide variety of till types?
- Materials unsorted – clay to boulder sizes?
- Slickensides in clay-rich till?
- Variable compaction and cementation?
- High permeability sands and gravels?
- Loess?
- Landslipped deposits?
- Creeping landslides?

Chapter 4

Planning, conducting and reporting of geotechnical investigations

4.1 THE NEED TO ASK THE RIGHT QUESTIONS

The French detective, Bertillon, considered by many to the father of modern crime detection, is reputed to have made the following statement:

> *"We only see what we observe, but we can only observe that which is already in the mind".*

Experience from analysis of many case histories (Stapledon, 1976, 1979 and 1983) shows that this principle (that we will only find that which we recognise) is equally true in engineering site investigations. In almost every foundation failure and contractual dispute over 'changed geological conditions', it is found that a major contributing factor has been the failure of project planners and site investigators to fully understand and define all of the geotechnical questions which needed to be answered by the site investigations. There are two types of questions, namely:

- *engineering questions*, which relate essentially to the design, construction and operation of any structure of the type proposed and
- *geological questions*, which arise from understanding of the site geological environment and its likely influence on the design, construction and operation of the project (see Chapters 2 and 3).

4.1.1 Geotechnical engineering questions

For dams which are intended to store water, or water plus solids, it is obvious that important questions must relate to the permeability, strength and compressibility of the foundations. However, there are many other equally important questions, because construction and operation of a dam causes much greater changes to a site environment than any other type of engineering activity. Five main processes involved in dam engineering, their principal effects on the site environment, and some resulting questions for the designer and site investigator, are as follows:

1. *Excavation*: To reach suitable levels for founding the dam, and also for the spillway, outlet works, other related works and for construction materials. Excavation causes removal of support from and increases in shear stress in the surrounding

material and hence raises questions of stability of the excavations themselves, during construction and/or operation, and of the stability of the adjacent valley slopes.
2. *Foundation loading*: Imposed by the dam structure, raises questions of compressibility of the foundation and its shear strength against sliding upstream or downstream, before and after filling of the storage and under flood and earthquake loading.
3. *Inundation – Filling the storage*: Causes changes to the groundwater regime, potential lowering of strengths of cohesive soils, weak rocks and joint cements and decreases in effective stress. These effects all add to the questions of stability of the dam and its foundation and also they raise the question of stability of the reservoir sides. These stability questions are more serious when water storage levels are required to fluctuate widely and rapidly.
4. *Operation of the reservoir during the life of the dam*: The water in the reservoir loads the dam, sets up seepage flows in the dam and its foundation, and potentially erodes the soil in embankment dams, and the soil and rock in all dam foundations. Reservoir levels vary during operation and include floods and drawdown to supply water to users.
5. *Flood discharges*: Have high potential to erode, raising questions about the location and erodibility of discharge areas for the spillway and outlet works.

Table 4.1 is a suggested checklist of geotechnical engineering questions to be answered during site investigations for new dam projects or for major upgrades of existing dams.

4.1.2 Geological questions

Typical geological questions for twelve common geological environments have been discussed and listed in Chapter 3. The following notes highlight the importance of asking and finding the answers to such questions, and of relating them to other site factors such as climate and topography and the proposed development.

4.1.2.1 *Questions relating to rock and soil types, climate and topography*

The relative importance of any one of the engineering questions on Table 4.1 and the amount and kind of site investigations needed to get the answer to it will depend on the topographic, geological and climatic environments in which the project is to be located. For example, consider the effects of first filling of a water supply dam in an arid region, in a steep sided valley underlain by very weak sandstone. The water table is likely to be very low or absent, and the sandstone may owe a large part of its strength and stiffness to cementation by a water-soluble minerals such as gypsum. Filling of the reservoir has the potential to cause dramatic changes to this site – significant rising of the water table, solution of water soluble mineral with resulting weakening and possible increase in permeability of the foundations, and possible instability in the storage area sides. Hence specific geological questions to be answered during investigations of this site would include the following:

- What are the cementing agents in the sandstone?
- How much reduction in strength and stiffness will occur in the sandstone when saturated for long periods?

Table 4.1 Geotechnical engineering questions.

1. Sources of materials, for the following purposes:
   – Earthfill, for the core or other zones.
   – Filters.
   – Rockfill.
   – Rip-rap.
   – Concrete aggregates.
   – Road Pavements.
   – For each material: Location of alternative sources, qualities/suitability's, quantities, methods for
   – winning and processing. Overburden and waste materials and quantities. Possible use of materials from required excavations, e.g. spillway, outlet works and dam foundations.
2. Reservoir
   – Watertightness.
   – Effect on regional ground waters – Levels or quality.
   – Stability of slopes inside and outside of reservoir rim.
   – Erodibility of soils – Possibility of turbidity problems.
   – Siltation rates and likely location of deposits.
3. Dam
   – Location – To suit topographic and geological situations.
   – Alternative* sites, for comparison of costs and of geotechnical and other issues.
   – Type(s) of dam suited to site(s)
   – Depths to suitable foundations for: concrete dam; earthfill; core; filters; rockfill; plinth or grout cap.
   – Nature of materials to be excavated, excavation methods, and possible uses of materials.
   – Stability of excavations, support and dewatering requirements.
   – Permeability, compressibility, shear strength and erodibility of foundations.
   – Foundations treatment(s) required: grouting; drainage; slurry concrete; dental treatment; filter blanket; other.
   – Embankment zones, methods of placement, and of control of quality, moisture and compaction.
   – Stability of dam, and dam plus foundation in all situations.
   – Monitoring systems: types, siting.
4. Spillway, river diversion works and permanent outlet works.
   – Location and type.
   – Excavation method(s), possible use for excavated materials.
   – Stability of excavations, need for temporary/permanent support.
   – Channel, need for lining/drainage.
   – Need for protection of the discharge area, or for excavation of a stilling basin.
5. Tunnels (for river diversion, spillway, or permanent outlet works)
   – Location of sites for portals.
   – Tunnel alignment(s).
   – Portals, excavation and support methods and final treatment.
   – Tunnel excavation, support and final treatment methods.
6. Seismicity of region.
   – Design earthquake, annual exceedance probability versus ground motion.
   – Maximum credible earthquake.

*Alternative sites for the dam and associated structures are usually investigated during stage 2 (Feasibility and site selection). See Table 4.3 for definitions of the suggested stages for the development of a dam project.

– Could solution effects during dam operations result in increase in permeability of the foundation?
– Could solution/strength reduction or water table rise result in instability in (a) the foundation or (b) the reservoir sides?

Table 4.2 Processes which may be active enough to affect a dam project.

– De-stressing	– Freezing
– Chemical weathering of rocks and/or soils	– Burrowing by animals
– Solution	– Growth of vegetation
– Deposition of cement	– Growth of vegetation
– Erosion by wind or water	– Rotting of roots of vegetation, or buried timber
– Deposition of sediment	– Seismicity, i.e. shaking, or displacement on a fault
– Creep, landsliding	– Vulcanism
– Subsidence	– Glaciation
– Pressure by groundwater	

At the other extreme, a site in a high rainfall area with gentle slopes underlain by very strong quartzite, and with a high water table, is likely to be almost unaffected by inundation.

It can be seen from these two very simple examples that certain generalizations can be made about geotechnical conditions likely to be met at a site, when its broad geological setting is known, and this is considered together with the site climate and topography.

4.1.2.2 *Questions relating to geological processes, i.e. to the history of development of the site*

It is not enough, during the design and construction of a major dam, to know simply what rock or soil types are present, their engineering properties and their approximate distribution. Understanding the site environment implies also understanding the geological processes which developed the region and the site. The most important processes are usually the youngest, commonly those relating to the near-surface. This is partly because they will have had a major influence on the strength and stability of the valley slopes, and also because they may still be active, or may be reactivated by the construction or operation of the dam. Table 4.2 is a list of such processes, some of which have been discussed in more detail in Chapters 2 and 3.

4.1.3 Geotechnical questions for investigations of existing dams

Table 4.3 lists geotechnical questions to be considered for investigations of existing embankment and concrete dams.

It is the authors' experience that embankment dams built in the 1950s to 1970s and even later have often not been constructed to the geometry shown on design drawings, or even to the as-constructed drawings. Also in many cases filters and transition zones have not been constructed as specified but have greater % fines and more plastic fines than specified. Where these are critical to the safety of the dam, as is often the case, it may be necessary to investigate the geometry of zoning and sample zones in the dam to confirm the actual situation.

4.2 GEOTECHNICAL INPUT AT VARIOUS STAGES OF PROJECT DEVELOPMENT

Most dam projects develop in five more or less distinct stages as set out on Table 4.4. Geotechnical input is required in all stages. Table 4.4 summarizes the broad objectives

Table 4.3 Geotechnical questions for investigations of existing dams.

Embankment Dams.
– Design loadings for the dam including floods and seismic.
– Geology of the foundation including identification of kinematically viable failure mechanisms in rock foundations and potentially liquefiable soils in the foundation.
– Embankment zones as designed and as constructed.
– Gradations and classification of materials in all zones in the embankment as specified and as constructed.
– Methods of placement, and of control of quality, moisture and compaction of all materials in the embankment.
– Permeability, compressibility, shear strength, and erodibility of foundations.
– Foundations treatment(s) which were carried out: grouting; drainage; slurry concrete; dental treatment; filter blanket; other.
– Monitoring systems: for deformations, pore pressures, seepage and reservoir level; the results of monitoring and surveillance from first filling to the present.
– Stability of dam, and dam plus foundation in all situations.
– Likelihood of cracking during construction, in service and under earthquake loads.
– Ability of the zoning and materials as constructed to control internal erosion and piping in the embankment, foundations and from embankment to foundation.
– Adequacy of the upstream and downstream slope protection to control erosion.

Concrete Dams.
– Design loadings for the dam including flood and seismic loads.
– Geology of the foundation including identification oaf kinematically viable failure mechanisms.
– Dam geometry, concrete properties and potential weaknesses including lift joints as designed and, as constructed, potential AAR effects.
– Methods of placement, and of control of quality of the concrete.
– Permeability, compressibility, shear strength, and erodibility of foundations.
– Foundations treatment(s) which were carried out: grouting; drainage; dental treatment; other and their durability and effectiveness in controlling seepage and uplift pressures.
– Monitoring systems: for deformations, pore pressures, seepage and reservoir level; the results of monitoring and surveillance from first filling to the present.
– Stability of dam, and dam plus foundation in all situations including seismic loading.
– Potential for erosion of rock foundations due to spillway releases causing undermining of the structure.

of the geotechnical work and the usual activities and reporting needed, during each stage.

It is important during the planning stages (Stages 1 to 3) that the site investigation studies should proceed in phase with and in close co-operation with other engineering work being carried out at those times, e.g. hydrological and topographic surveys, design and preparation of specifications. This is because most decisions affecting the project design or feasibility from the geotechnical viewpoint will affect the work requirements in the other fields. Conversely, decisions made from the results of these other studies will affect the scope of the geotechnical work.

It is essential that the dam designer takes a keen interest in the geotechnical work and that field and office consultations with the geotechnical team are frequent. Design drawings (e.g. cross-sections through structures) should include the main features of the foundation (i.e. the portion which Nature has made) as well as details of what Man proposes to build.

It is also vital that the geotechnical investigator be given adequate time to do his or her work. The whole process of geotechnical investigation, and indeed of design,

Table 4.4 Geotechnical input during stages of development of dam project.

Stage No.	*Name*	*Objectives and activities (Geotechnical)*
1	Pre-Feasibility	Assist in the selection of possible sites and obtain enough understanding of the geological situation to plan the feasibility and site selection studies. Usually includes review of existing data plus a short ground and sometimes air inspection.
2	Feasibility and site selection	Assess the project feasibility and design from the geotechnical viewpoint, considering both the regional and local geological situations. Explore alternative sites for dam and other key structures, and adopt the most promising sites. Explore these further if necessary to confirm feasibility and provide sufficient data for preliminary design and feasibility stage cost estimate. Provide regular progress reports and prepare formal report at the end of Stage 2 with a definite statement confirming (or otherwise) the project feasibility from the geotechnical point of view.
3	Design and specification	Answer any questions outstanding or arising from the feasibility studies, and additional geotechnical questions raised during the design. Further site exploration and testing are usually necessary. Provide regular progress reports, and report for tenderers or construction agency. Provide assistance in the design of the embankment and other structures and preparation of the specification.
4	Construction	Ensure that the geological picture exposed during construction is as assumed in the design and if not, that modifications are made to the design, if necessary. Provide day to day advice on geotechnical matters to the Resident Engineer. Provide record of geological exposures during construction and of any rock movements, water inflows, etc. in regular progress reports. This data is vital to the Surveillance Group (see Stage 5); should any malfunction develop it may form the main basis for (sometimes rapid) correct action. Activities include detailed mapping, colour photography, review of inspector's records, installation and reading of instruments, monitoring simple tell-tales, etc. Input to design and specification modifications to suit site conditions. Geologist/geotechnical engineer team leader must be part of a foundation inspection group, should such a group be convened.
5	Operation	Ensure that the structure is performing as designed, from the geotechnical point of view, and assist in the design of remedial measures, if it is not. Inspect the completed structure, the site area and records from instruments and operator's observations, at regular intervals, preferably as a member of a surveillance committee, including representatives of design, construction and operation branches. Should any malfunction be evident, assist in design of remedial measures. This could involve conducting further site investigation and/or analysis of the geotechnical records from Stages 1 to 5.

requires 'thinking', not just the mechanical production of drill hole logs, pit logs and the like. Too often, time is reduced by the project planners to a minimum, without realising the risk that action could put the whole project in.

Timing and co-operation are just as critical during the construction stage. During this stage the foundation rock or soil is exposed better than ever before or (hopefully) after and, if the site investigator arrives too late, the exposed area showing a critical clue can be covered by the first layer of fill or concrete. It goes without saying that unless the site investigator has the confidence and co-operation of both the construction team and the design team, this phase of the work will not be as fruitful as it otherwise could be. In extreme cases, lack of a competent geotechnical observer and advisor during the construction stage can prove disastrous – resulting in either expensive contractual disputes or later failure of the structure.

Similarly, during the operation stage inspections must be made at regular intervals, to ensure that any malfunctions are discovered while there is still time to remedy them. Such inspections are carried out as part of a surveillance programme as discussed in Chapter 20.

4.3 AN ITERATIVE APPROACH TO THE INVESTIGATIONS

During each of the project planning stages (Table 4.4, Stages 1 to 3) the geotechnical studies should follow the iterative approach shown on Figure 4.1, which is modified from ISRM (1975) and Stapledon (1983). This applies to investigations for both new dams and on existing dams.

The following notes relate to the four phases on Figure 4.1.

Phase 1. First, the objectives of the work, or questions to be answered, are defined. As discussed in Section 4.1, these will include both geotechnical engineering and geological questions. The number of geological questions which can be defined clearly at this time might be quite small, if little is known of the geology of the region or site.

Phase 2. Existing geological, geotechnical and other data relevant to the site are collected and compiled to give a tentative geotechnical model (or models). Tentative answers to the questions asked in Phase 1 are obtained where possible, from local knowledge or from rapid analyses, or both. New questions are usually added during this phase, arising from the understanding acquired while assessing the existing data.

Phase 3. The investigations are planned to confirm the tentative answers and tentative geotechnical model and to answer the outstanding questions. The plan is based on the 'broad to particular' approach and types of activities set out in Section 4.4.

It is usual (and desirable) for the work to proceed in stages. For each stage the proposed activities are related to time and money respectively, in an activity chart and cost estimate. The proposals are set out in a report to the client, seeking approval to proceed.

Phase 4. The investigations proceed in stages as planned. A cooperating team of engineering geologist(s) and geotechnical engineer(s) is able to complete most of the studies in either logical sequences or concurrently and to integrate their results. Answers to the purely geological and geotechnical engineering questions are produced progressively from the results, by combinations of analyses and judgements.

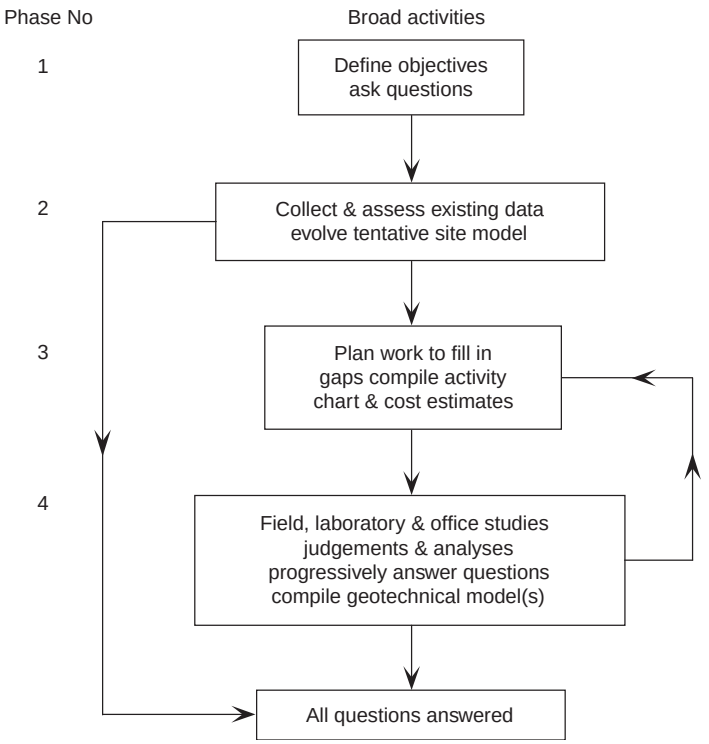


Figure 4.1 Activity flow in site investigations.

Sometimes, new questions arise and extra work has to be planned and approved (see arrow back to Phase 3). The site geotechnical model, and in some cases more detailed models for particular project features, e.g. dam, spillway, outlet works and quarry, are compiled. It will be vital throughout this phase for the design engineer to be closely involved with these investigations, not only to be fully aware of the outcome of these studies, but also to foresee design changes that would help the project's development.

When all questions have been answered with sufficient confidence, the investigation is complete. Until then, further cycles of investigation are carried out until the required levels of confidence are reached. The principle of 'diminishing returns' is applied.

4.4 PROGRESSION FROM REGIONAL TO LOCAL STUDIES

The geological studies in Phases 1 to 4 start with consideration of the site location in relation to the global tectonic situation, and should include study of the geology of a

broad region surrounding the site. This is necessary to assess the effects on the project of large scale processes, some of which (Table 4.2) may have potential to damage it.

The regional geological studies are followed by geological and geotechnical engineering studies at and near the site, on intermediate and detailed scales. The objectives and usual activities of the regional and detailed studies are set out below, generally following ISRM (1975). More detailed descriptions and discussion of the various activities are presented in Chapter 5.

4.4.1 Broad regional studies

4.4.1.1 *Objectives*

To provide an understanding of the geological history of the project area, that is, of the processes which have developed the present geological situation at and in a broad region around the site. In particular it is important to determine the following:

1. Major geological processes which have the potential to cause impact on the project.
   - Any processes that are active or potentially active, e.g. as listed on Table 4.2.
   - The possible effects of any active processes on the proposed works both during construction and in service.
   - Whether any construction activities (e.g. excavation) or operation of the storage are likely to cause such changes to the existing regime (e.g. stress or hydrologic) as to require remedial works or to affect the project feasibility.
2. To determine the regional stratigraphy and geological structure.
3. To explain the geomorphology of the project area in terms of the regional stratigraphy, structure and geological history.
4. To draw attention to important features, e.g. major faults or landslides, occurring at or close to the site, but not exposed or recognisable at the site.
5. To get an appreciation of the regional groundwater conditions.
6. To form a logical basis for the location and proving of sources of construction materials.

4.4.1.2 *Activities*

The amount and money spent on regional studies will depend upon the size and complexity of the project, its consequence of failure, and the amount and relevance of regional information already available. The activities usually will include some or all of the following:

1. Examination of existing regional geological maps, cross sections and reports.
2. Interpretation of satellite images and aerial photographs.
3. Interpretation of ground photographs.
4. Ground reconnaissance over previously mapped areas and remapping of important areas with the project objectives in mind.
5. Compilation of regional plans and cross sections showing the proposed project works.

4.4.2 Studies at intermediate and detailed scales

4.4.2.1 *Objectives*

1. To explain the development of the site topography in terms of the regional and local geology and geological history.
2. To delineate any major features, e.g. folds, faults or landslides, which may or may not be indicated already by the regional studies.
3. To provide geotechnical models for the site and for particular project features, and to provide answers to the geotechnical engineering questions.

4.4.2.2 *Activities*

1. Geotechnical mapping on intermediate and detailed scales.
2. Studies of the pattern of defects using structural geology and stereographic methods.
3. Application of geophysical methods (commonly seismic refraction) and interpretation of the results in the light of the evolving geological picture.
4. Planning and technical supervision of direct exploration, which can include core (or other) drilling, sluicing, and excavation of pits, trenches, shafts or tunnels.
5. Geotechnical logging and photographing of the cores of drill holes and of exposures in the exploratory excavations.
6. Logging of drill holes using video imaging, cameras, impression devices, etc.
7. Permeability testing in drill holes.
8. Field and/or laboratory testing of soils.
9. Field and/or laboratory testing of rocks.
10. Plotting of all geological and test data, *as soon as it is obtained*, onto plans, sections and 3-dimensional models (physical or computer generated). In this way it is possible to visualise and analyse the surface and subsurface data, progressively filling in gaps in the subsurface picture until the required degree of detail is obtained. All plans, sections and models show the proposed works, at least in outline. The end results of this activity are the geotechnical model and geotechnical design parameters.
11. Assist in the design and analysis to assess the behaviour of proposed structures and their foundations, under construction and operating conditions.

It is the authors' experience that too many investigations fail to follow this progression from regional to intermediate and then to detail scale and as a result critical information is missed and/or the geological context of the site is not understood.

Too often investigations of existing dams are carried out without the involvement of engineering geologists, especial one with the relevant experience and there is a resultant lack of understanding of the foundations of the dam.

Potentially more damaging is the lack of collaboration between the dam designer and the engineering geologist during the site investigations which sometimes occurs. This can be on occasions a result of over-commitment on other projects.

With the advent of computer drafting it often happens that data is not plotted onto plans as the data is collected but left until after the investigations are completed. This results in gaps in site investigations, drill holes drilled too short or in the wrong position and waste of resources as equipment has to be re-established to fill the gaps.

4.5 REPORTING

It is important during all stages, that the geotechnical facts, interpretations, conclusions and decisions made from them are recorded regularly by a system of formal progress reports. A comprehensive report is essential at the end of each stage, setting out the answers to the questions of that stage and with recommendations for the next stage.

In general terms, a geotechnical report should consist mainly of a carefully planned and ordered sequence of drawings (and some photographs) with relatively short explanatory text, and appended tables and calculations. The drawings and photographs should convey a clear three-dimensional picture of the surface and subsurface conditions at the site, and the geotechnical properties of the foundation and construction materials. A clear distinction should be made between factual data and inferences made from them. The report (or a separate one) should include the results of analysis and design, e.g. stability analysis, design of filters.

A formal system for checking and certification is needed for all drawings and reports.

4.6 FUNDING OF GEOTECHNICAL STUDIES

In the opinion of the authors, it is most important that dam projects receive adequate funding at the feasibility stage. This is because the question 'Is it economically feasible to build and maintain a safe structure at this site?' is the really vital and difficult one. Usually there are alternative sites but finally one must be adopted and the above question must be answered in relation to it.

Unfortunately, some organisations may not have (or fail to seek) sufficient amounts of money until they believe they have shown a project to be feasible. Also, they may not wish to, or cannot, acquire the site until feasibility is assured. Because of these matters, and the inevitable pressure from landowners and conservation groups, the investigations at this vital stage often tend to be less logical and less thorough than necessary – too much reliance is placed on indirect methods (geophysical) and drilling of small diameter boreholes, and/or not being allowed to drill in the river bed because of environmental constraints. The advantage of these methods is that they cause minimal disturbance of the land. However, without an adequate understanding of the geological situation, which often can be obtained only by large, continuous exposures in deep bulldozer trenches or access track cuttings, the results of geophysical studies and drilling can be difficult to interpret. For major, high hazard dams at geologically complex sites, exploratory adits are often the most appropriate method for answering the feasibility questions.

The consequences of a poorly planned or inadequately funded feasibility stage investigation can be:

- Adoption of the less satisfactory of two alternative sites.
- Adoption of a site which later proves to be non-feasible or for which large cost over-runs occur to address unforseen conditions (for examples of such sites see Table 3.4).
- Abandonment of a site (or possibly a whole project) which would have been proven feasible, by well conceived (but more expensive) studies.

The authors consider that the 'observational method' as adopted by Terzaghi & Leps (1958) at Vermilion Dam, should not be used as a means of overcoming funding difficulties during the feasibility and design stages of dam projects. They agree with the practice of excavating part or all of the dam foundations during the design stage, or prior to awarding the main construction contract (Hunter, 1980), but believe that this should be done only when all questions relating to feasibility of the project have been adequately answered.

4.7 THE SITE INVESTIGATION TEAM

The following seven attributes for success in site investigations for dams, put forward by Stapledon (1983), suggest that a team approach is necessary, especially for large dam projects:

1. Knowledge of precedents.
2. Knowledge of geology.
3. Knowledge of soil and rock mechanics.
4. Knowledge of geotechnical, dam, and civil engineering design.
5. Knowledge of civil engineering and dam construction.
6. Knowledge of direct and indirect exploratory methods.
7. Above average application.

It should be clear from Section 4.1 to 4.4, that Attributes 1 to 5 are necessary for understanding, defining and answering the geotechnical questions associated with dams.

Attribute 6 refers to direct subsurface exploratory methods (e.g. pits, boreholes) and indirect methods (e.g. geophysical traversing). Knowledge of the application and limitations of methods such as these is necessary if the questions posed are to be answered effectively and economically.

The authors consider that these six attributes are necessary for successful site investigations for dams. They also consider that these attributes can be provided most effectively by a team including engineering geologists and engineers. The dam designer must play his or her part co-operatively in this important phase of the work. It is important that all team members have enough 'general' knowledge in all areas not only to be able to communicate effectively with one another and with engineers involved with the design, construction and operation of dams, but also actually to do so. As well as having this broad knowledge of the industry, it is desirable that they have Attribute 7, 'above average application' as defined by Stapledon (1983) "*. . . ability to get things done; this involves effective cooperation with others, enthusiasm and drive and the ability to make decisions in the field.*"

On larger projects it is common to have a Peer Review Panel which reports to the Owner. It is important that the review panel is involved in reviewing the planned investigations, the investigations including site visits and the reports.

Chapter 5

Site investigation techniques

Several techniques or 'tools' may be used in a dam site investigation designed to follow the broad framework explained in Chapter 4. This chapter discusses some of the most common techniques, their applicability and limitations. It is emphasised that use of several techniques is always required, as restriction to a single investigation method would be unlikely to yield correct answers to the site questions in an economical manner.

5.1 TOPOGRAPHIC MAPPING AND SURVEY

A fundamental requirement for the investigation and design of any project is accurate location and level of all relevant data. Topographic maps at suitable scales are essential with establishment on site of clearly identified benchmarks. All features recorded during the investigation should be located and levelled, preferably in relation to a regional coordinate system and datum. A local system may be established provided that at some stage during the investigation the relation between the regional and local survey systems is determined.

Survey control for regional studies can utilise Global Positioning System (GPS) techniques but detailed project studies should be based on standard ground survey methods.

These statements may appear obvious but, in many instances, interpretation, construction and related contractual problems have been shown to have originated from poor survey control.

Topographic maps at several scales are required. The actual scale will depend on the size and complexity of the area but the following common scales are given as a guide:

- Regional maps, 1:250,000 with 20–50 m contours to 1:25,000 with 10 m contours.
- Catchment area, 1:25,000 with 10 m contours to 1:2000 with 2 m contours.
- Project area, 1:1000 with 2 m contours to 1:200 with 1 m contours.
- Individual engineering structures, 1:500 with 1 m contours to 1:200 with 0.5 m contours.

Regional topographic maps issued by government agencies in most countries are published at several standard scales which range from 1:250,000 with contours at

20–50 m to 1:25,000 with contours at 10 m interval. The user of these maps should consider their original purpose and the accuracy of the information plotted on them in relation to the project requirements. The notes on the map which indicate the method of compilation; whether aerial photogrammetry, Lidar or ground survey; and the date of preparation of the map may be relevant.

Photogrammetry can be used in the preparation of project specific plans provided that the aerial photographs have been flown at low level and that accurate ground control points can be identified on the photographs and that vegetation is not dense.

Photogrammetry is often not adequate in steep, tree-covered areas. The authors are aware of two projects where errors of up to 20 m in elevation have occurred due to inadequate allowance for tree cover. This resulted in the requirement for substantially larger saddle dams than estimated in the feasibility studies and significantly affected the viability of the projects.

Lidar is a remote sensing technology that measures distance by illuminating a target with a laser and analysing the reflected light.

Airborne Lidar can be used to create DTM (Digital Terrain Models) and DEM (Digital Elevation Models). This is quite a common practice for larger areas as a plane can take in a 1 km wide swath in one flyover. Greater vertical accuracy of below 50 mm can be achieved with a lower flyover and a slimmer 200 m swath, even in forest, where it is able to give you the height of the canopy as well as the ground elevation. A reference point is needed to link the data in with the survey datum.

The authors' experience is that Lidar is a very valuable method particularly for contouring large areas such as the area surrounding a reservoir. The contours can be used to delineate the extent of large landslides for example, as well as the detailed geometry of the slide. Likewise, Lidar would appear to have some distinct advantages when it comes to survey of potentially inundated areas in dam-break studies and the estimation of downstream consequences. This activity would be more likely related to the investigations of an existing dam.

The field survey team should liaise with the site investigation group, and provide a series of clearly labelled ground control marks throughout the project area to assist in the location of features identified during the geotechnical mapping. All boreholes, pits and trenches should be located, levelled and clearly labelled.

5.2 INTERPRETATION OF SATELLITE IMAGES AERIAL PHOTOGRAPHS AND PHOTOGRAPHS TAKEN DURING CONSTRUCTION

Satellite image maps and aerial photographs are available from government agencies in most countries. For investigations of existing dams, photographs taken during construction are an invaluable aid to assessing the geology and construction of the dam.

5.2.1 Interpretation of satellite images

Standard LANDSAT images are at 1:100,000 scale but images at 1:500,000 and 1:250,000 are also available.

These small scale images or image maps provide a broad view of the region in which the project is to be located. This broad view can indicate correlations between geological features or the position of geological boundaries or faults, when these features are of such great extent that they are not recognisable on larger scale photographs covering smaller areas.

The broad view also provides an indication of relationships between the regional geology and landforms, drainage, soils, vegetation and land-use, which may be useful in:

- planning of access routes to and within the project area,
- location of potential sources of construction materials and
- assessing reservoir siltation rates.

5.2.2 Interpretation of aerial photographs

5.2.2.1 *Coverage*

If the existing aerial photographs in the project area are inadequate in quality, coverage or scale, it is best to take new photographs with the following advantages:

- the photographs will show present conditions,
- the required scales and coverage can be specified and
- the photographs can be used to prepare topographic maps.

Aerial photographs taken at different dates can indicate changes in site conditions.

5.2.2.2 *Interpretation*

Photo-geological interpretation using a stereoscope forms a major part of the initial appraisal of regional and local site conditions during the pre-feasibility and feasibility stages (see Section 4.4.1. and Table 4.4). It usually gives a good indication of likely geotechnical constraints on the project and the extent of investigations required.

Much has been written about the techniques of photo-geological interpretation and their application in engineering (Ray, 1960; Rib and Liang, 1978; Rengers and Soeters, 1980; Bell, 1983b; Lillesand, 1987). Comments will therefore be limited to aspects of special significance in dam engineering.

The stereoscope provides a three-dimensional image with a vertical scale which is exaggerated by a factor which depends on the distance between the photo-centres. This exaggeration can be of benefit as it highlights surface features which are actually more subtle, but interpreters should realise that the ground slopes viewed under the stereoscope appear steeper than they really are.

A major advantage of aerial photography is that distance of observation is not impeded by relief. It is possible to observe features on both sides of a hill at the same time and thus establish their continuity, if it is present. This is particularly important in the interpretation of geological structure.

Landforms which reflect the structure of folded rocks show up well on aerial photographs. In horizontal strata, mesa forms are common and, in dipping strata, dip slopes and scarps indicate the direction and dip of the bedding (Figure 5.1).



Figure 5.1 Aerial photograph showing folded sedimentary rocks in Central Australia.

An important part of photo-interpretation for dam engineering is the recognition and plotting of lineaments. Lineaments are simply linear features or linear arrangements of features that are visible on the photographs.

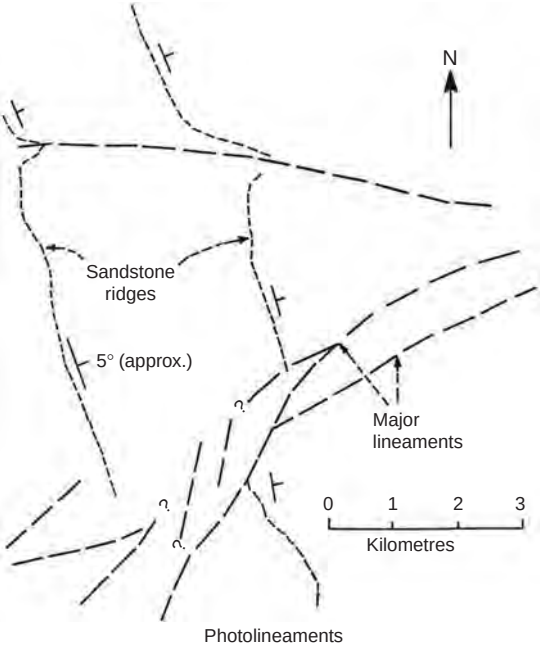
Faults, particularly those which are steeply dipping, usually show up well as lineaments. A lineament indicative of a fault may be a linear arrangement of features which are topographically low, or otherwise indicative of the presence of deep soils or low strength materials. Such features include straight section of rivers or creeks, gullies, saddles, springs, swamps and usually dense vegetation.

Figure 5.2 shows several lineaments passing through an area in Central Australia underlain by sandstones dipping to the east at about 5 degrees. The major lineaments occur along sheared zones with little or no displacement of the beds. The minor lineaments visible on the aerial photograph follow joints.

Linear arrangements of other features, e.g. vegetation boundaries, are usually found to correlate with geological features.

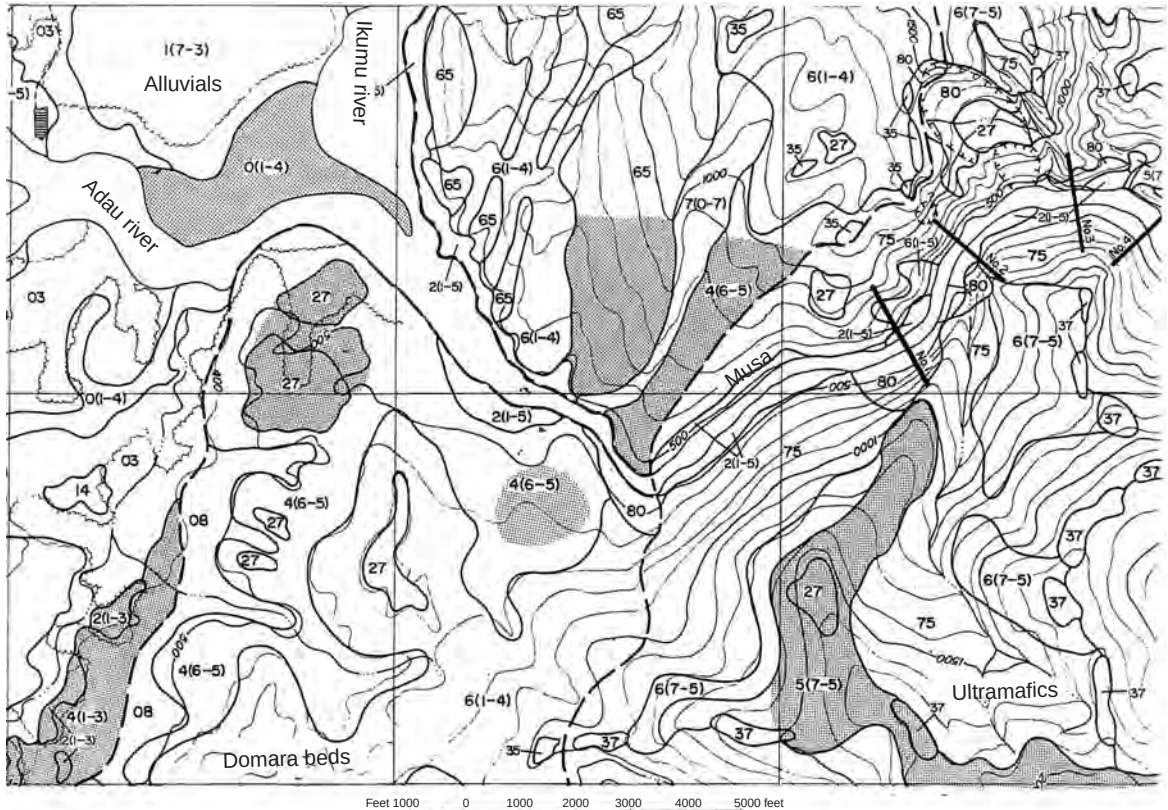


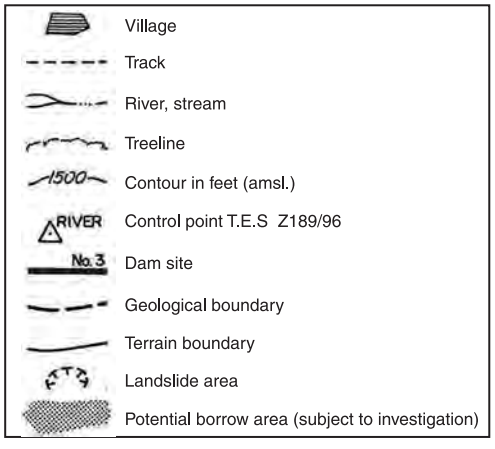
(a) Aerial photograph



(b) Sketch showing lineaments

Figure 5.2 Aerial photograph and sketch showing lineaments in an area underlain by gently dipping sandstones.


Alluvials
Ikumu river
Adau river
Musa
Domara beds
Ultramafics
Feet 1000 0 1000 2000 3000 4000 5000 feet
Metres 300 0 300 600 900 metres
Contour interval 100 ft.



Note: The base map is complied from CDW Drawing Nos. Ph68/70 & Ph67/225. The reliability of the map position on the grid, north of 335,000N, is fair.

Terrain classification framework & explanatory notes

Geology		Terrain mapping units					
Geological group	Lithology		No.	Physiographic feature	Natural slope range	No.	Soils	U.S.C. Symbol
Ultramafics: [(Kgd) & (Kpd) undifferentiated]	Massive noncumulus peridotite, dunite – commonly layered and in places brecciated: minor gabbro.	Horizontal or Sub-horizontal surfaces	0	Level or depressional, poorly drained areas.	Level or very low gradient.	0	Rock outcrop or skeletal soils.	–
			1	Level, very gently undulating or very gently sloping surfaces–well drained areas.	Level or up, to 2°.	1	Gravels – (alluvial or colluvial).	GC-GM-GW
Domara beds: [includes domara River beds (Qpd) and minor Musa Volcanics (Qpm)]	Conglomerate, siltstone, shale with Minor intermediate to basic agglomerate, tuff and basic lava.		2	Undulating or irregular surfaces.	Variable–predominantly long, gentle slopes (to 8°).	2	Sand – uniform profile.	SW-SM-SP
			3	Strongly undulating, irregular or hummocky surfaces.	Variable–predominantly short, moderately steep slopes (8°–18°).	3	Sandy or silty medium textured soils (gradational, uniform or duplex profile).	SM-ML or SC-CL
Alluvials: (Qa)	Unconsolidated gravel, sand, silt and clay, local areas of boulder alluvium.	Sloping surfaces	4	Slopes gentle (hill slopes; foot slopes etc.)	To 8°	4	Silty soils or stratified soils – predominantly silty.	SM-ML-MH
			5	Slopes moderate, (hill slopes, escarpment slopes etc.)	8°–18°	5	Gravelly or stony clay, silty clay, sandy clay etc.	GC-CH & MH
			6	Slope steep (hill slopes, escarpments slopes etc.)	18°–30°	6	Duplex soils – medium to coarse textured soil over fine textured soil.	SP-SC/CL-CH
			7	Slopes very steep	30°–45°	7	Clay soils – uniform gradational or duplex (including glay clays).	CL-CH or CH
			8	Precipitous slopes, cliffs, benched slopes, escarpment slopes etc.	>45°	8	Clay soils or soils with very high liquid limit. (clays, silty clays, volcanic ash soils etc.).	CH or MH
			9	Water ways.		9	Organic soils–peat, peaty clay etc.	OL-OH-Pt

Example: Geology — Ultramafics — Geological group; Terrain mapping unit — 75 — Physiographic feature; Soil*

* Note: A dual symbol (1–5) indicates that both material types 1 and 5 are likely to occur within the unit as mapped.

Figure 5.3 Terrain map for construction materials investigation, Musa Dam, Papua New Guinea (courtesy of Coffey Geosciences).

As discussed in Chapter 2, Section 2.10, it is very important that evidence of any currently active or past landsliding in the project area is recognised early. Fortunately the presence of active or ancient landslides is usually indicated by:

- Un-vegetated scarps, cracks and areas of exposed soil or rock,
- characteristic topographic forms, e.g. scarps, spoon-shaped troughs, hummocky or steep ground and areas of internal drainage,
- areas of anomalous vegetation, e.g. where trees are dead, or younger than elsewhere, or where the vegetation is more dense due to an area of deeper or wet soil,
- evidence of past or current restriction or damming of streams.

Rib and Liang (1978) provide guidance on the recognition of landslides on aerial photographs in a range of geological environments.

In densely vegetated areas, evidence of active or very recent landsliding may not be readily visible on photographs. For example, in some tropical areas shallow landsliding produces subdued topographic forms and revegetation is rapid. As shown in Figure 3.34 large landslides show up well even in tropical rainforest. Fookes *et al.* (1991) describe how examination of several sets of aerial photographs assisted the analysis of slope movements at Ok Ma dam site, Papua New Guinea.

Another use of photo-interpretation is in terrain analysis where the features observed, including topography, geology, soils, drainage and vegetation are used to divide the area into land units with similar characteristics (Grant 1973, 1974). This type of study is more suited to regional assessment of catchment or reservoir areas or to the location of construction materials, than to localised assessments like those for dam sites. Figure 5.3 is an example of a terrain map prepared to assist in the location of construction materials. Such maps form a logical basis for the planning of ground surveys followed by exploration of possible borrow areas. These maps should not be relied on for selection of sites without a field check of site conditions.

5.2.3 Photographs taken during construction

Any review of an existing dam should include a thorough search for photographs taken during construction, and systematic review of the photographs. They are particularly valuable as a record of foundation preparation and clean-up and for assessing matters such as segregation of filter and transition zones.

5.3 GEOMORPHOLOGICAL MAPPING

Before embarking on a subsurface investigation program, which may be (a) expensive and (b) provide limited information of dubious relevance, it is recommended that the surface evidence should be systematically recorded.

The ground surface reflects both the underlying geology and the geomorphological development of the area. Geomorphological mapping of surface features can provide an indication of the distribution of subsurface materials, their structure and areas of possible mass movement, e.g. landslides. Geomorphological mapping should be carried out at two levels – an overall appraisal of the study area and a detailed recording of site data using a topographic map as the base. Hutchinson (2001), Varnes (1978) and Brunsden *et al.* (1975) describe mapping methods.

The overall appraisal involves an examination of the topographic features of the area, surface shape, drainage, local depressions and areas of distinctive vegetation.

An hour spent on a convenient hilltop reviewing the geomorphology of the whole area before commencing detailed mapping may provide an understanding of the problem which is obscured in a closer examination. Binoculars enable closer examination of more remote localities, and land-based stereo-photographs have proved a useful method of recording information for subsequent re-evaluation in the office.

The compilation of a geomorphological map as a basis for the succeeding stages in the study involves the identification and location of all relevant features including changes in slope, scarps, cracks, areas of seepage and displaced or rotated vegetation.

These features are recorded on the topographic map using one of the many systems of symbols which have been developed. Most of the systems have a similar basis but differ in detail depending on the site conditions experienced in the original study. The symbols shown in Figure 5.4 are often sufficient and commonly used. Any geomorphological map should be accompanied by a comprehensive legend explaining the symbols.

Of particular relevance is the demarcation of boundaries of areas judged to be disturbed by slope movements. Figure 5.5 is a large scale geomorphological map of a landslide area above the abutment of O'Shannassy Dam.

5.4 GEOTECHNICAL MAPPING

Geotechnical mapping is essentially geological mapping aimed at answering engineering questions. At any particular site it involves the location and plotting on suitable scales of all data which assists in understanding the geotechnical conditions at that site.

5.4.1 Use of existing maps and reports

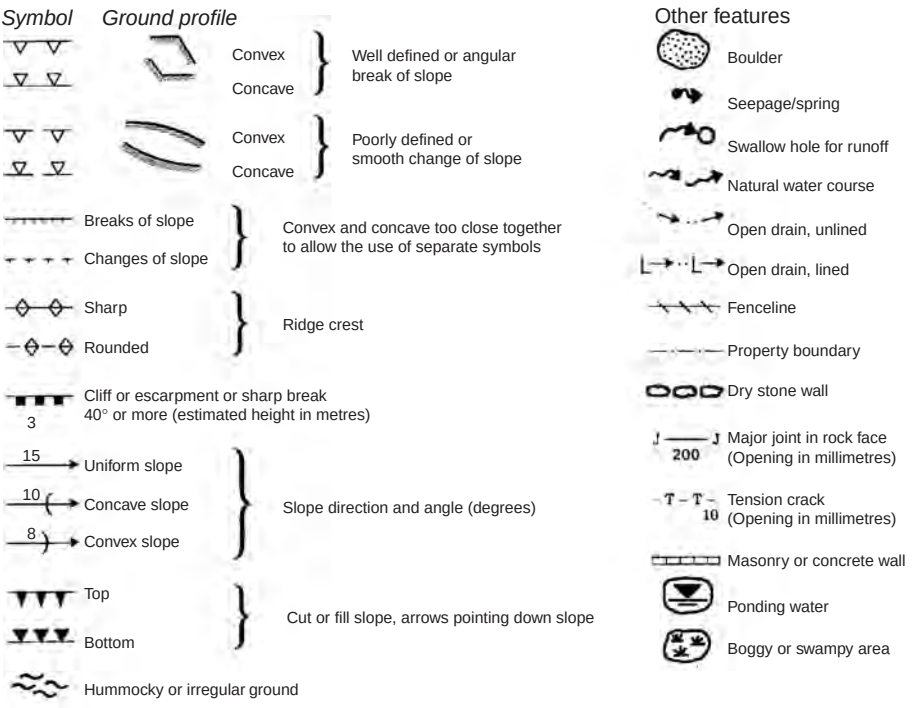
Some useful data can often be obtained from existing maps and reports prepared for other purposes.

Maps showing the regional geology on scales ranging from 1:100,000 to 1:1,000,000 are usually available from government agencies. Some mining areas and areas of existing or proposed urban development may have been mapped at larger scales. The regional maps are often accompanied by explanatory notes which are useful.

The regional maps show the inferred distribution of the main rock types and the inferred geological structure. Areas where rock is overlain by unconsolidated sediments, surface soils or talus are not differentiated unless these deposits are known to be widespread and of significant depth. The data plotted on the maps is obtained from examination of surface outcrops, information from stratigraphic and exploration drilling and air photo interpretation.

The main value of these maps is providing an understanding of the stratigraphy, structure and geological history of the region, i.e. the broad geological environment in which the project is to be located. As set out in Chapter 3, this understanding can provide a useful insight into the range of geotechnical conditions which might be

Topography



Example of use of topographic symbols:

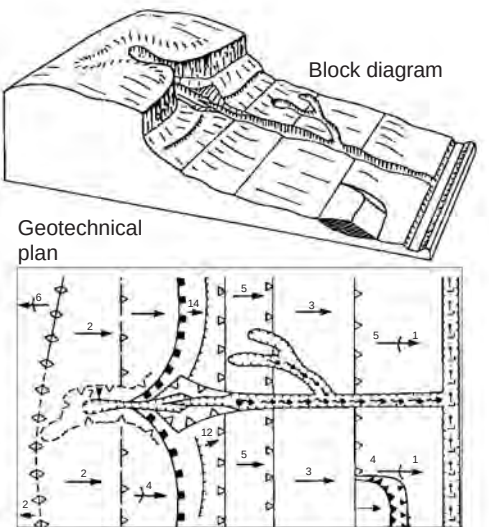


Figure 5.4 Geomorphological Mapping Symbols (Courtesy Jeffery and Katauskas).

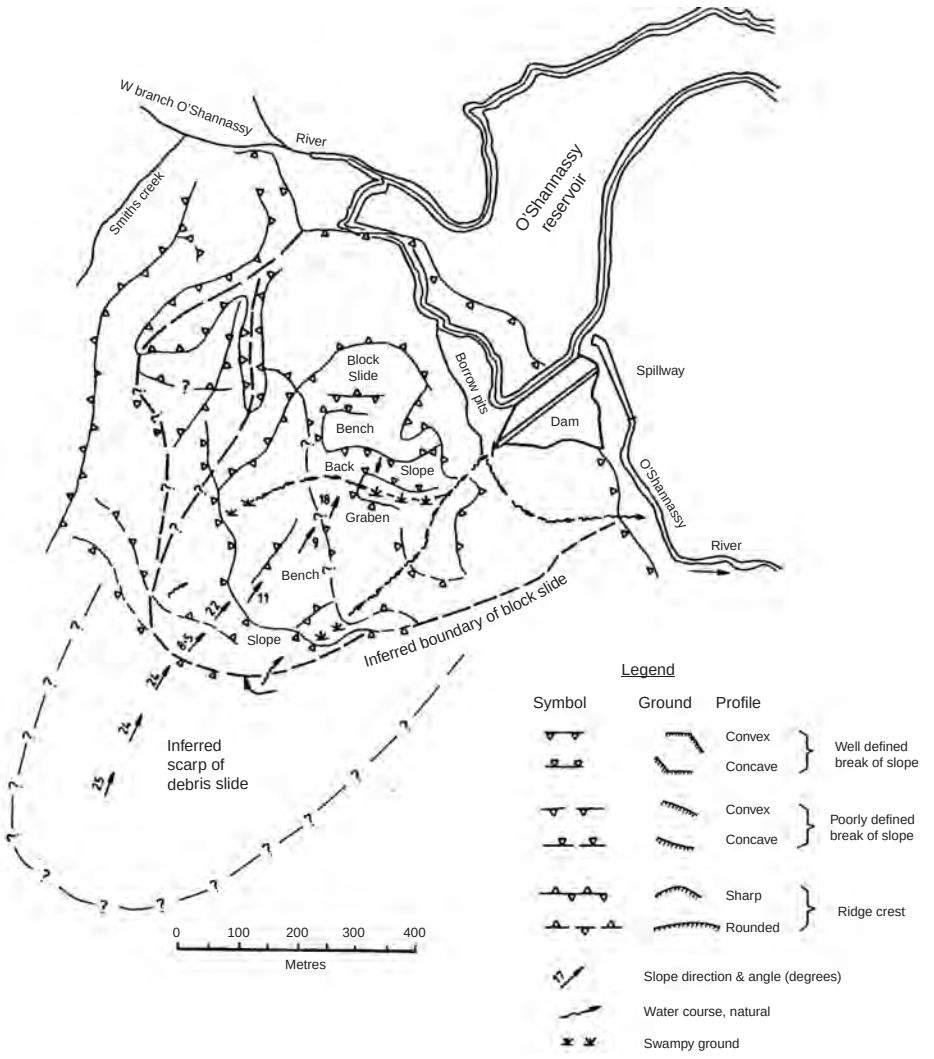


Figure 5.5 Large scale geomorphological plan of landslide area at O'Shannassy Dam, near Melbourne, Australia (courtesy Jeffery and Katauskas, and MMBW).

expected at the project sites. The maps are also useful during studies associated with access routes, material sources and reservoir and catchment areas.

Local deposits of alluvial, colluvial or residual weathered materials are usually ignored or briefly mentioned on regional geological maps. Because of this the maps generally do not answer detailed questions related to dam projects.

Geological reports prepared for different purposes, e.g. for mineral exploration, can provide some useful information.

Regional maps showing distribution of soil groups, classified for agricultural purposes, may also be available. These maps may be a useful supplement to the regional geology map, particularly in the search for construction materials.

Land capability maps, based on the regional soil surveys with particular attention to potential landsliding and erosion, may be available for limited areas.

5.4.2 Geotechnical mapping for the project

5.4.2.1 *Regional mapping*

When published regional geological maps are available they are usually able to provide the regional geological understanding required for a dam project. For major dam projects, and for all projects where these maps are to be used in access, material or reservoir studies, they should be checked on the ground and on air photos. Where necessary they should be updated by the addition of data of engineering importance. Such data will include lineaments, landslides, scarps, swamps, springs and areas of problem soils.

If no satisfactory regional geological map is available, and the proposed dam is of major size and high hazard, then it would be prudent to prepare a regional map specifically for the project.

However produced, the regional geological plan as used and included in the project reports should show the location of the proposed works and the outline of the proposed storage.

5.4.2.2 *Geotechnical mapping*

Geotechnical mapping at and near the sites of the proposed works is the key to the success of the site investigation. The mapping involves the identification and location of all surface features relevant to the establishment of geotechnical models at the sites. The plans and sections produced by the mapping form the initial geotechnical models. They are the basis for the subsurface exploratory work and are aimed at checking the models, filling in gaps and answering specific questions raised by the indicated geological environments (see Chapters 3 and 4).

The geotechnical maps are usually produced at an intermediate scale (1:5000 or 1:2500) covering the general works sites, and at 1:1000 or 1:500 covering the immediate area of the sites. The maps show the following types of factual information as shown on Figure 5.6.

- ground surface contours,
- geomorphic features, e.g. slope changes, areas of hummocky ground,
- geological surface features, e.g. areas of rock outcrops, scree, boulders and soil,
- features of *in situ* rock, e.g. rock types and their boundaries, attitudes of bedding and foliation, the nature, location and orientation of important geological defects such as sheared or crushed zones,
- groundwater features, e.g. springs, seepage, areas of swamp and vegetation indicating moist or wet ground,

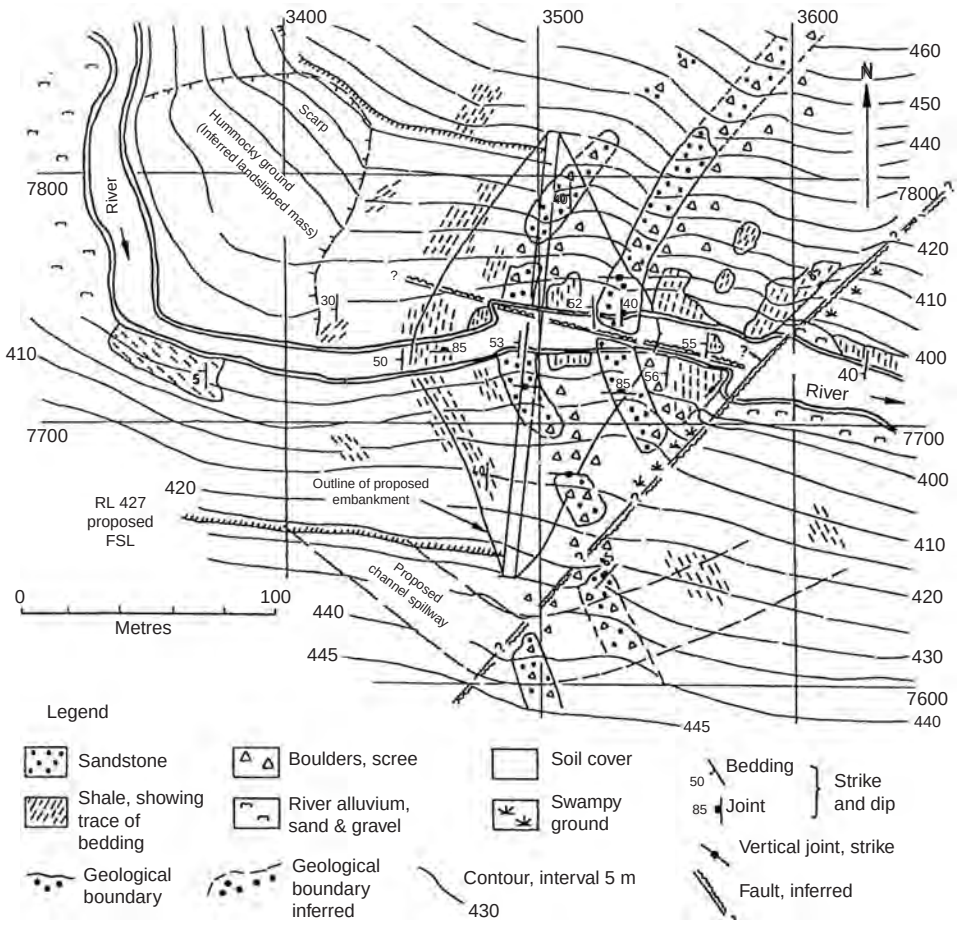


Figure 5.6 Plan showing some of the features presented on a large scale geotechnical plan of a site for an embankment dam.

- the location of tracks, roads, test pits and trenches, with summary logs of the soils and rocks exposed,
- the position of drill holes and geophysical traverse lines,
- the proposed works in outline including the full supply level of the proposed storage.

Figure 5.6 is an example of the first stage of compilation of a geotechnical plan at the site for a concrete faced rockfill dam about 35 m high. At this stage the plan shows a factual record of surface geological and geomorphological features. Three relatively important features are inferred – the landslide upstream from the site and the two faults. Other important features might also be suspected, for example bedding-surface

faults at the boundaries for the sandstone beds, because no actual shale-sandstone boundaries are exposed in outcrop, even at river level.

It should be noted that such a plan would normally be prepared on 1:500 or even 1:200 scale on an A2 or A1 size sheet. For the sake of legibility for publication at its present size the plan has been simplified greatly, i.e. it contains much less geological detail and wider contour spacing than would usually be present.

Position identification is important during the mapping. In sparsely vegetated areas the combination of enlarged air photos and contour plans at the same scale may enable positioning to an acceptable level of accuracy. Where required more accurate control can be achieved by ground survey methods or by use of GPS.

5.5 GEOPHYSICAL METHODS, SURFACE AND DOWNHOLE

Geophysical methods have been extensively used in dam investigations, both on dam construction projects and in the assessment of the condition of existing dam structures. There are many different methods; the selection of the method(s) relevant to a particular problem will usually depend on the regional and local site conditions, the nature of the problem, timing and cost.

The *advantages* of the use of geophysical methods include:

- They are non-invasive and can be carried out from the surface or from existing boreholes.
- They can provide information on site conditions between data points, e.g. boreholes.
- They may be able to identify local areas of concern which have no surface expression, e.g. cavities, areas of concentrated groundwater flow.
- The surveys can usually be performed quickly and cover a relatively large area.
- Recent development of computer analysis and presentation of results (e.g. tomography) has assisted interpretation.

The *disadvantages* of the use of geophysical methods include:

- Each method measures a particular physical subsurface property which may or may not be relevant to the problem under examination.
- Some borehole geophysical methods are affected by the presence of steel casing.
- Each method requires specialist, possibly expensive, equipment and skilled operators.
- The results involve the recording and analysis of a great amount of numerical data.
- The mathematical analysis is based on an assumed subsurface model which may be different from the actual geological situation.
- Much of the data involves the averaging of information.
- The accuracy of the results may be lower than required.
- The interpreted subsurface profile usually needs to be confirmed by drilling.
- Establishment of the equipment and operators is expensive and also stand-by time waiting for other operations, e.g. drilling.

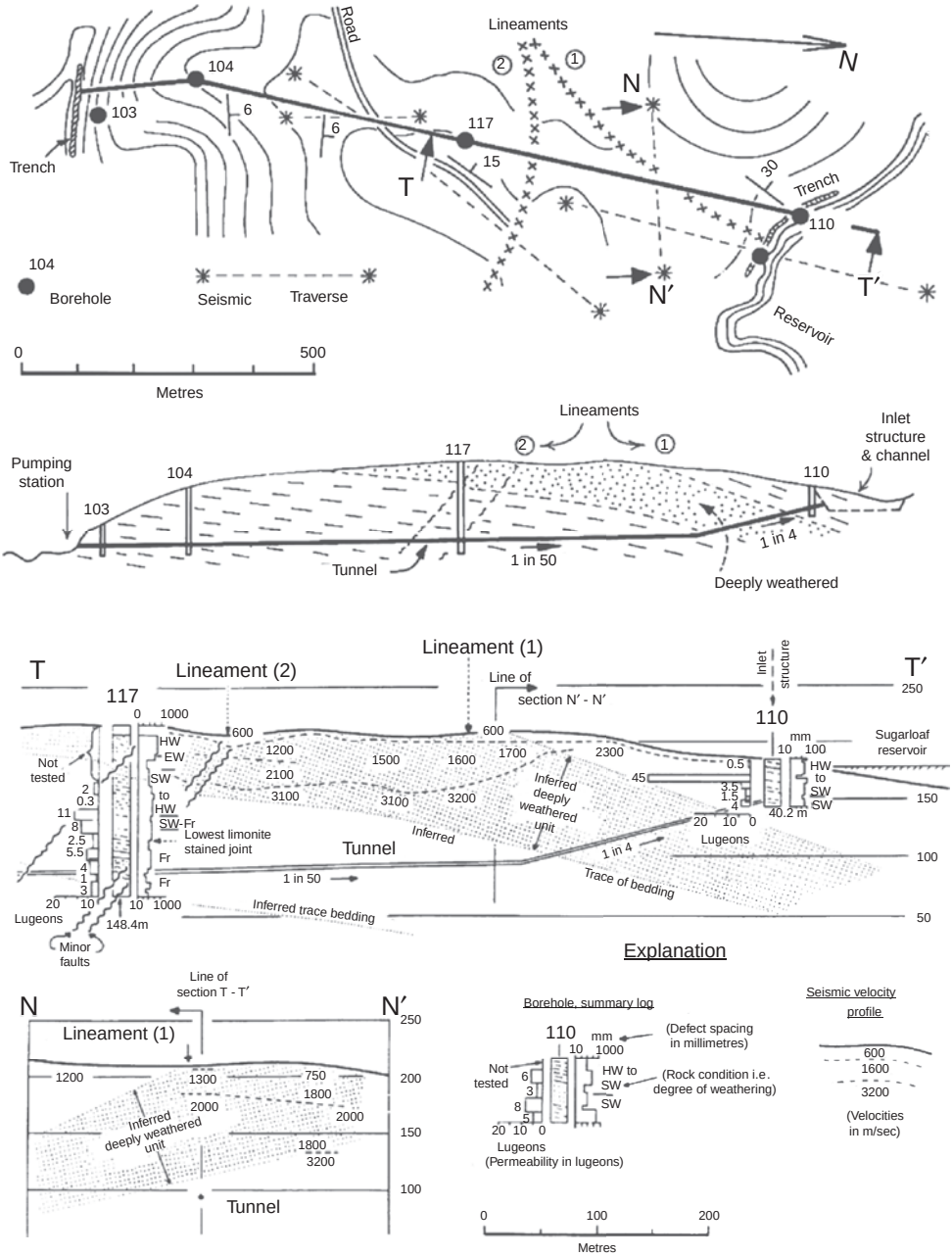


Figure 5.7 Section along Sugarloaf Inlet Tunnel showing seismic refraction profiles and deeply weathered unit (Stapledon, 1988b).

Table 5.1 Application of surface based geophysical methods.

Type of Survey	*Applications*	*Limitations*
Seismic Refraction	Determines compression "P" wave and shear wave "S" wave velocities depending on the energy source used. Mostly "P" wave. From these can be interpreted depth to rock, and properties of the overlying soil and weathered rock.	May be unreliable unless strata are thicker than a minimum thickness. Velocities must increase with depth; velocity inversions cannot be detected. Difficult to interpret if rock profiles are irregular or stepped. Information represents average values. "P" wave velocity of saturated soils are all the same, and to differentiate "S" wave data is required.
Multi-channel analysis of surface waves (MASW)	Determines shear wave velocities based on dispersion of Rayleigh waves to depths up to 30 metres. Not constrained by requiring velocities to increase with depth.	Relatively sophisticated analysis which may produces complex results which sometimes cannot be explained by other subsurface data.
Seismic Reflection	Produces geological stratigraphic and structural data using seismic waves similar to sound waves on shore or acoustic air blast off shore.	Difficult to recognize the arrivals of the reflected waves from refracted and direct surface waves for shallow investigations. Mostly applicable where bedrock depth is >30 m. Seldom useful for dam investigations as a result.
Self-potential (SP) profiling	Uses the current generated by flow of water (the streaming potential) in soils or into open cracks to locate seepage areas and concentrated flows, e.g. into open cracks in dams or reservoir. The only technique that directly indicates subsurface water flow.	Accuracy affected by presence of ferrous objects. May need to be accompanied by a magnetic survey.
Electrical resistivity	Measure the resistivity of soils and fractured rocks to locate boundaries between materials with contrasting resistivity; e.g. between clean granular and clay strata, fractured and non-fractured rock, degree of saturation, position of water table and soil-rock interface. Resistivity imaging may locate cavities.	Sometimes difficult to interpret and subject to correctness of the assumed subsurface conditions; does not provide any indication of engineering strength properties. Accuracy affected by presence of ferrous objects.
Electromagnetic conductivity profiling (TDEM)	Locates boundaries between clean granular and clay strata; assessment of degree of saturation, position of water table and soil-rock interface. More rapid reconnaissance than electrical resistivity	Sometimes difficult to interpret and subject to correctness of the assumed subsurface conditions; reliant on resistivity contract; does not provide any indication of engineering strength properties. Accuracy affected by presence of ferrous objects.
Magnetic	Locates ferrous objects. Used in combination with self potential and resistivity surveys.	Does not locate other metals.

(*Continued*)

Table 5.1 Continued.

Type of Survey	*Applications*	*Limitations*
Microgravity	May be used to delineate the boundary between alluvial soils and rock, including paleo-channels. Where seismic refraction methods cannot be used; e.g. in densely populated urban area; locate small volumes of low density materials and larger cavities.	Use of expensive and sensitive instruments in rugged terrain may be impractical. Requires precise levelling.
Ground-penetrating Radar	Provides a subsurface profile, may locate cavities, buried objects, boulders and soil-rock interface	Success is site specific and depends on having a contrast in dielectric properties. Has limited penetration particularly in clay materials and laterites.

Areas of investigation where the *correct use* of geophysical methods has provided valuable information include:

- Delineation of boundaries between the underlying *in situ* rock and transported materials such as alluvium, colluvium, glacial debris and landslide debris.
- Delineation of boundaries between residual soil, weathered rock and fresh rock.
- Delineation of boundaries between sandy and clayey soils.
- Location of anomalous foundation features, e.g. igneous dykes, cavities, deeply weathered zones, fault zones, buried river channels.
- Assessment of rippability, depth of foundation excavation, depth of cutoff excavation, liquefaction potential.
- Location in existing structures of seepage paths, low density zones, cavities.

It is the authors' experience that no surface geophysical method has been able to contribute to the location of thin, weak seams in the rock mass. This information is often an issue of great importance for any dam, especially for concrete dams.

The different geophysical methods and their application to dam engineering are briefly discussed below with reference to papers containing detailed method descriptions. Further discussion is included in Whiteley (1983, 1988), Stapledon (1988b), McGuffey *et al.* (1996), Fell *et al.* (2000).

FWHA (2003), USACE (2005b) and Sirles (2006) give more details of the methods and their interpretation, application and limitations.

5.5.1 Surface geophysical methods

The application and the limitations of the different geophysical methods which can be carried out from the surface without the aid of boreholes (other than to assist interpretation) are summarised in Table 5.1.

5.5.1.1 *Seismic refraction*

This method utilizes the fact that seismic waves travel at different velocities in different materials; in rock and soil masses the velocity increases with increase in

substance strength and compactness. Whiteley (1988) provides details of the method and interpretation of the results.

Profiles of apparent seismic 'P' wave velocity are produced as shown on Figure 5.7, a section along the line of Sugarloaf Reservoir Inlet Tunnel. Air photo interpretation had shown two lineaments crossing the line of the tunnel. Seismic refraction traverses in combination with trenching and core drilling indicated that the lineaments are the surface expressions of a deeply weathered rock unit and a minor fault. This interpretation was confirmed during the driving of the tunnel.

Seismic refraction using the 'P' waves is the method most commonly used for delineation of boundaries between soil and weathered rock, and within weathered rock profiles. It is the authors' experience that, in a residual weathered profile, the base of the lowest velocity layer is usually a reasonable approximation of the probable general foundation stripping level for an embankment dam, and the base of the second layer is a reasonable approximation of the cutoff excavation level.

When combined with geological information about the rock mass, 'P' wave velocities can be used to estimate rock rippability (MacGregor *et al.*, 1994). Seismic velocities alone should not be used for predicting rippability.

Some limitations and requirements of the method are:

- The method cannot distinguish between sandy and clayey soils, and between soil and weathered rock where the boundary is gradational.
- The accuracy of the method is affected by poor velocity contrast between 'layers' and where there are 'cliffs' in the rock profile.
- The geophysical 'model' assumes sub-horizontal layering or boundaries. Results may be misleading where this is not so.
- The method cannot detect velocity inversions, i.e. high velocity material above low velocity material (unless cross hole or down hole techniques are used, see Section 5.5.3).
- Large energy input (usually explosives) is required in loose material.
- Accurate ground survey is essential for good results.

5.5.1.2 *Self potential*

The self potential or SP method (streaming potential) is based on the measurement of the natural potential difference which exists between any two points on the ground under groundwater flow or flow into open defects such as cracks in concrete dams or open defects in the reservoir floor formed by differential settlement. The method has little application for new dam construction projects but has been extensively used to locate areas of leakage from existing dams and reservoirs (Ogilvy *et al.*, 1969; Corwin, 1999; Revil and Boleve, 2007; Boleve, 2013).

5.5.1.3 *Electrical resistivity*

The electrical resistivity method measures the resistance of the ground to induced electrical current. Interpretation assumes a horizontally layered model. Whiteley (1983) and Corwin (1999) provide details of the method.

The method has been used to locate fractured and fault zones, zones of deep weathering and cavities (McCann *et al.*, 1987; Smith and Randazzo, 1987). It can

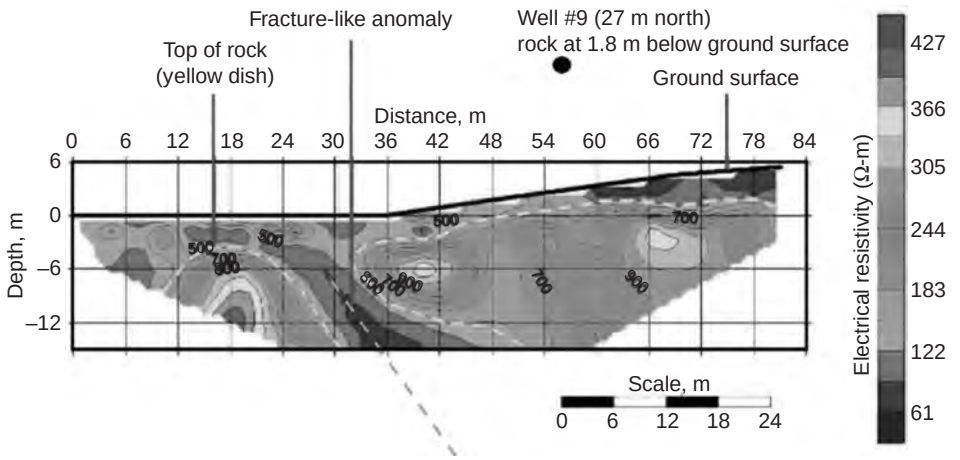


Figure 5.8 Resistivity data in fractured siltstone and shale FHWA (2003).

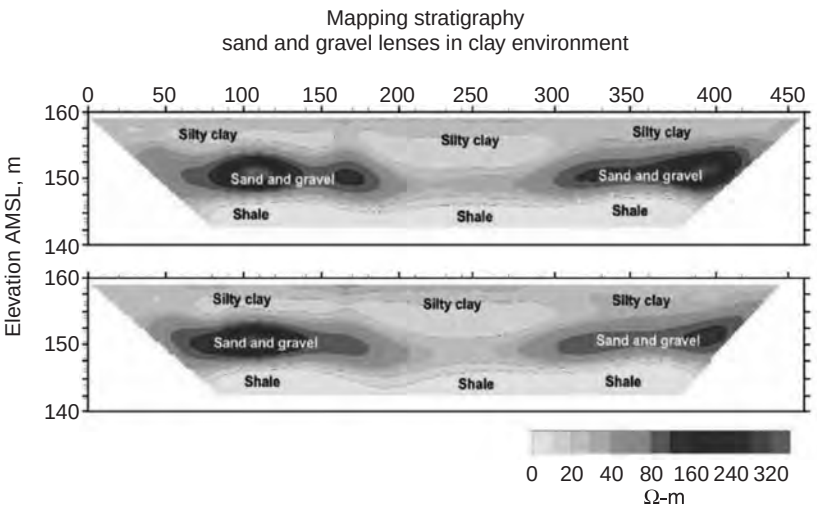


Figure 5.9 Resistivity data showing stratigraphy of soils overlying rock FHWA (2003).

also be used in the exploration of alluvial deposits where permeable gravel and sand beds can be distinguished from low permeability clays or rock. This capability has been applied in searches for construction materials beneath alluvial terraces and for foundation investigations at dam sites where significant alluvial deposits occur.

Figures 5.8 and 5.9 give examples of resistivity data.

At dam sites in valleys the results of resistivity surveys are affected by the irregular terrain and by changes in the electrical properties between dry materials on the abutments and the wet material beneath the valley floor. In situations where the valley

floor width is less than five times the thickness of alluvial material there are significant side effects that produce misleading results.

These limitations and the generally poor depth resolution (about 10% of depth) indicate that the resistivity method has limited application in dam site investigations. It has however been successfully used in the review of some existing embankment structures (Corwin, 1999).

5.5.1.4 *Electromagnetic conductivity*

EM surveys have been widely used in North America (Corwin, 1999) in the assessment of existing embankments. The development of commercial software packages for the interpretation of EM data has considerably assisted the analysis of the data. Care has to be taken to select the correct frequency and to ensure that the geophysical interpretation matches the likely geotechnical model. EM is affected by above ground metal objects and by AC noise sources, e.g. power lines.

The transient electromagnetic (SIROTEM) method produces resistivity-depth profiles (Whiteley, 1983). The method is quicker and requires less space than the resistivity method, but may not be suited to the shallow depths normally associated with dam foundations.

5.5.1.5 *Magnetic*

Surveys using a sensitive magnetometer do not provide significant detail on the subsurface profile but can indicate the presence of buried ferrous objects, e.g. pipelines that can have a major effect on EM and resistivity traverses.

5.5.1.6 *Microgravity*

Accurate measurement of the earth's gravitational field, together with tomographic processing of the data, can indicate the presence of anomalous subsurface structures e.g. dykes and cavities.

The survey procedure is simple but requires precise location and level of each measuring point, which can make the method tedious, particularly in rough terrain.

Whiteley (2005) gives examples of the use of the method to determine the boundary between alluvial soils and rock, including location of paleo-channels.

5.5.1.7 *Ground penetrating radar*

Ground penetrating radar (GPR) uses the ability of VHF electromagnetic (radio) signals to penetrate through soils and rocks. Usually the waves are radiated from an antenna mounted on a wheeled sled. Reflections are received from features such as fractured zones and cavities in rock, and from distinct layers in soils and the rock surface. The method has also been used down hole.

The depth of penetration depends on the frequency of the signal with lower frequency giving greater penetration but less resolution.

The method works best for shallow unsaturated soils free of silt or clay, overlying rock.

The method has been claimed to achieve some success in the location of sinkholes and cavities in karst areas and this appears to be its main potential in dam engineering.

The results of surface surveys in karst areas are described by Wilson and Beck (1988). Cooper and Ballard (1988) also describe the location of cavities at the El Cajon dam site in Honduras, using a down-hole technique.

Limitations of the method include the need for a near regular surface to permit passage of the sled and very limited depth penetration (<1 m) particularly through clayey soils.

Siggins (1990) and Morey (1974) describe details of the method.

5.5.2 Down-hole logging of boreholes

Table 5.2 summarizes the available methods for down-hole logging of boreholes.

Profiles of electrical resistivity, self potential, gamma-ray emission and neutron absorption are routinely obtained during exploration for oil and coal, where most boreholes are uncored. Geophysical logging of each hole produces graphical plots from which the characteristic features of the different lithological formations can be recognised (Whiteley, 1983; Whiteley *et al.*, 1990). The development of commercial software packages for the interpretation of the data has considerably assisted its analysis.

The logging commonly requires uncased and water or mud-filled holes. The equipment is specialised and potentially expensive and surveys have to be coordinated with the drilling programme.

Dam site investigations involve many shallow holes and nearly all holes are sampled or cored. Geophysical logging can assist in correlation between boreholes, the location of potential leakage zones and in the assessment of the properties of valley floor sediments. In the opinion of the authors in many cases the additional cost of retaining the equipment on site may not be justified by the benefit of the additional information obtained. However in some situations some techniques are valuable e.g. borehole imaging can provide invaluable data on joint and bedding parting openings greatly complementing the logging of core, and water pressure testing.

5.5.3 Cross-hole and up-hole seismic

The use of seismic waves in combination with boreholes has provided a range of methods to identify subsurface conditions both in the dam foundation and within the dam structures. In cross-hole seismic a hydrophone array is lowered into one borehole and an energy source – a specially developed drop hammer – into a second borehole. By changing the location of the source and also the position of the hydrophone array the arrival times of the direct and refracted seismic waves can be computed into a tomographic image of the section between the two boreholes.

An alternative method using a single borehole involves the placement of the hydrophone array in the borehole, with the energy source at different points on the surface. The results can be computed to provide information on a cone of material up to 100 m deep with a radius up to twice the depth of the hydrophones.

The methods are described by Joyce *et al.* (1997), Whiteley (1998) and Van Allerand Rodriguez (1999).

Table 5.2 Applications and limitations of borehole logging methods (adapted from Fell *et al.*, 2000 and McGuffey *et al.*, 1996).

Method	*Parameter Measured*	*Applications*	*Limitations*
Ultra-sonic log	Determination of sonic velocities, attenuation of shear velocity in shear zones, or imaging the sides of the borehole wall	Shows fractures, other discontinuities	Requires uncased hole. Image less clear than borehole camera
Thermal profile	Temperature	Zones of inflow (lower/ higher temperature water)	Open hole not necessarily required
Caliper log	Borehole diameter	Infers lithology by borehole enlargement by erosion during drilling by drill fluid	Requires an uncased hole
Induction log	Electrical conductivity	Infers lithology by clay content, permeability, degree of fracturing	Lower resolution than resistivity log, but can evaluate unsaturated zone and PVC cased boreholes
Resistivity	Electrical resistivity	Infers lithology, (particularly between sand and clay)	Applicable only in saturated zone; requires open hole
Self potential	Electrical potential from mineral reaction and groundwater flow	Infers lithology, oxidation/ reduction zones, subsurface flows	Applicable only in saturated zone; requires open hole. Data difficult to interpret.
Natural Gamma	Natural Gamma radiation	Infers presence of clay and shale	Mud coating on borehole wall may affect results
Gamma Gamma	Natural density	Provides log of density, from which lithology may be inferred	Provides only density. Health and safety issues
Neutron Neutron	Moisture content (above water table). Porosity (below water table)	Log of moisture content, from which lithology or wetter zones may be inferred	Provides only water contents. Health and safety issues
Borehole Camera or Video e.g. RAAX	Visual image of fractures and structure	Can identify structure. May be able to interpret dip of fractures and opening of joints, and bedding partings	Requires uncased hole. Images affected by water quality, smearing of sides of hole
Acoustic reflectivity	Acoustic reflections	Can identify lithology and structure. May be able to interpret dip of fractures and orient cores	Requires uncased hole

5.6 TEST PITS AND TRENCHES

5.6.1 Test pits

Test pits excavated by a rubber-tyred back-hoe or tracked excavator are effective in providing information on subsurface conditions in dam foundations or existing dams, for the following reasons:

- They are relatively cheap and quick.
- The subsurface profile is clearly visible and can be logged and photographed.

- Material types, the nature and shape of their boundaries and structure can be observed and recorded in three dimensions.
- The absence or presence of groundwater is indicated and the sources of inflows can usually be observed and their flow rates recorded.
- Undisturbed samples can be collected.
- *In situ* tests can be carried out.
- The resistance to excavation provides some indication of excavation conditions likely to be met during construction.

It may be possible to leave pits open for inspection by design engineers and representatives of contractors. In this case fencing is usually required. The presence of fissures in clays can be assessed by leaving the pit walls to dry for some time, e.g. overnight. Fissures can be classified using the systems shown in Figure 6.29.

Figure 5.10 shows a log of a test pit which includes a sketch of one wall with descriptions of each material type, traces of their boundaries, and traces of defects. Some limitations of test pits are:

- Pit depth is limited by the reach of the machine and its ability to dig the material. The maximum depth of a pit dug by backhoe is 4 m but the machine commonly refuses on very weak rock. Larger excavators can reach to depths of more than 6 m and, depending on the nature of the rock defects, can excavate into weak rock.
- Local laws on health and safety support requirements of excavations must be followed. Collapse of pit walls occurs commonly, particularly in wet ground. The effective use of steel or timber support (Figure 5.11) or benching can enable safe access to pits for logging and sampling.
- Groundwater inflows may cause excessive collapse and limit effective excavation, even with supports.
- Test pits disturb the local environment, but if carefully backfilled and the site revegetated their long-term impact is minor.
- Test pits can result in defects within the embankment or foundation if the backfill soils are not properly compacted.

5.6.2 Trenches

A logical extension of the use of test pits is the excavation of trenches, using either a tracked excavator or bulldozer and ripper. Such trenches provide virtually continuous exposures of the subsurface materials at sites where there is little natural outcrop. Careful cleanup by small backhoe or hand tools is usually required to expose defects, particularly in rocky materials. Trench exposures are logged in a similar manner to test pits as shown in Figure 5.10 and Figure 5.12.

Following the slope movements which caused disruption of construction of Tooma Dam (as described in Chapter 2, Section 2.10.3.2), clear evidence of past slope movements were exposed as infilled extension features in cut faces of access tracks. It was concluded that this evidence would have been found, and probably understood, if the site investigation had included bulldozer trenches.

Following that example the sites for many dams built in Australia have been explored extensively by bulldozer trenches which have been designed to provide the answer to geotechnical questions and also access for drilling rigs. More recently,

SMEC

GEOTECHNICAL LOG OF EXCAVATION

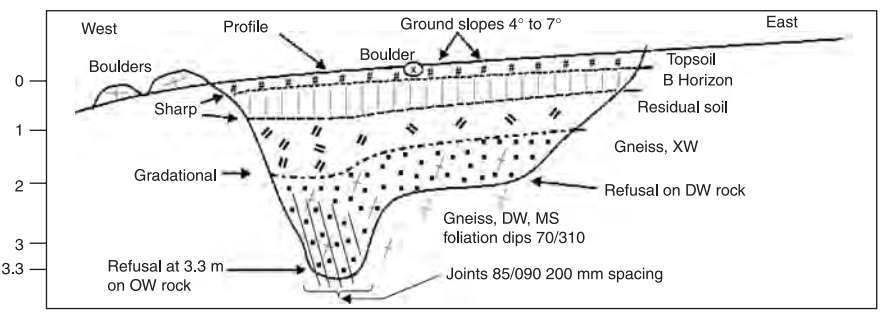
Client: Antarctic energy

PIT No: WS 23
Sheet No.: 1 of 1

Project:	Weddell Sea dam	Logged by:	Jim Wilson	Project No:	SP1112
Feature:	Spillway	Checked by:	Mike Smith	Surface elevation (m):	403.5
Location:	Crest	Contractor:	Seadrill	Angle from horizontal:	–90°
Easting (m):	31770	Commenced:	30.3.01	Direction:	West-East
Northing (m):	59560	Completed:	30.3.01	Depth to groundwater (m):	Dry

Elevation / depth (m)	Substance description	Graphic log	Soil class	Moisture	Density	Consistency	Sample	Test type	Result	Structure and origin
	Silty sand, fine to medium, black, many roots to 2 mm diameter 0.35 m		SM	D	L					Topsoil, contains boulders
403	Clay, high plasticity, mottled grey and brown, some medium sand 0.8 m		CH			V St	D	H	300	B Horizon, blocky; Fissures 1.90;1.9.3
1	Clayey sand/Sandy clay sand medium-coarse, brown		SC - GL	M		V St	D			Residual soil; contains many roots and tubes up to 5 mm diameter
402	Grades to 1.6 m									
2	Clayey silty sand, medium to coarse brown; sharp boundary with rock at 1.9 m at East end and 3.3 m at West end		SC - SM Rock		VD					Gneiss, extremely and distinctly weathered; shows foliation and joints as shown on sketch
401										
3	3.3 m									
	End of pit at 3.3 m – Refusal on distinctly weathered gneiss									

Sketch of excavation: Looking North



Notes: Timber support obtained from Mawsons Hul.

Basis of descriptions and details of abbreviations are given on Explanatory Notes		Project No:	SP1112
Contractor: Seadrill	Digging method: Excavator, 1000 mm bucket	PR No:	WS 23
Equipment: Penguin 200	Support: Timber	Sheet No:	1 of 1

Figure 5.10 Log of test pit.

the use of long bulldozer trenches has been frowned upon because of environmental concerns.

These trenches have totalled more than 3 km in length at each of the following large embankment dams – Talbingo, Dartmouth, Sugarloaf and Thomson.

The effectiveness of well prepared trenches in providing an understanding of dipslope stability at the Sugarloaf site is described in Chapter 2, Section 2.10.3.4.

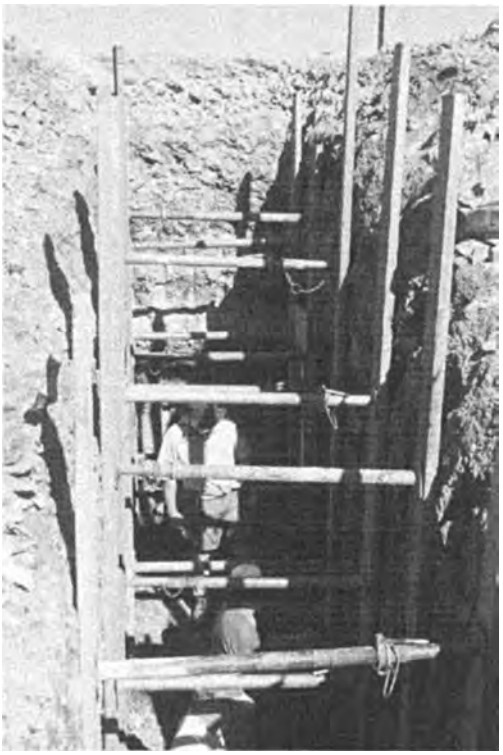


Figure 5.11 Timber supported trench/pit in Happy Valley Dam, Adelaide, South Australia (courtesy SMEC and SA Water).

A major benefit of trenching is the experience gained from the use of plant of similar size and type which may be used during construction. It is good practice to record machine and ripper types, material types and excavation rates.

At sites for large structures in geologically complex areas with little natural exposure, trenching can be the most practical method of providing the answer to vital feasibility and design questions. A disadvantage of trenching is that it causes much disturbance of the site but, as with test pits, trenches can be backfilled and revegetated if construction work does not proceed.

5.7 SLUICING

In some situations during site investigation the bulk removal of soil by slicing using high-pressure water jets can expose the rock surface in the dam foundation and thus extend the information obtained from natural exposures on rock condition and structure.

Sluicing is an effective option on steep, rocky slopes where the soil is relatively thin and sandy. Stiff to hard clay is difficult to sluice. Sluicing requires a powerful pump

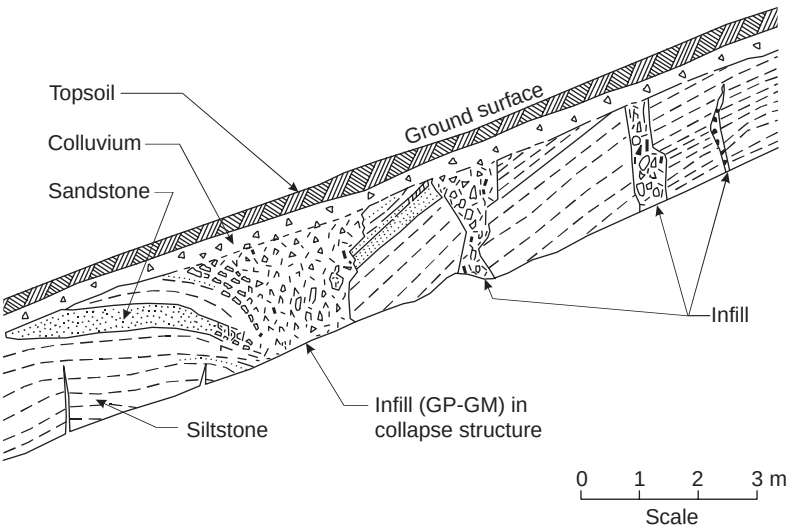


Figure 5.12 Part of log of trench, Sugarloaf Dam showing infill features resulting from slope movement (Regan, 1980).

and careful control of the operation to limit environmental effects. The movement of the mud slurry produced by the sluicing has to be controlled and contained using sedimentation basins.

Sluicing has been used with success at the Little Para Dam (South Australia), Gordon Dam (Tasmania) and Upper Ramu diversion dam (Papua New Guinea). Clearly, for environmental reasons there could be much resistance to use sluicing nowadays.

5.8 ADITS AND SHAFTS

In investigations for a major dam it may prove necessary to investigate a part or parts of the site, e.g. an area underlain by cavernous limestone or disturbed by a landslide, in more detail than can be achieved by the combination of surface excavations and drilling.

In such situations the excavation of adits or shafts into the area of concern provides the opportunity for:

- direct observation of the ground conditions,
- measurement of orientation of defects and comparison with surface measurements,
- *in situ* testing,
- underground investigation drilling.

Exposures in adits and shafts should be logged and photographed in detail using the same general approach as for test pits and trenches. All exposed faces, including the floor if practicable, should be logged.

Adits and shafts are expensive and slow to excavate. They are likely to require support, ventilation and drainage.

The requirements for, and location of, adits or shafts should be carefully considered in relation to all the available design and geotechnical information. It is necessary to justify the expense by the clarification of site conditions which they may provide. Wherever possible exploratory adits should be incorporated in the design of the dam, e.g. as drainage or grouting galleries.

Shafts using 1000 mm or larger diameter augers or clam-shell excavators are sometimes used to explore alluvial and glacial deposits.

5.9 DRILL HOLES

5.9.1 Drilling objectives

The main objectives of drilling are to extend the knowledge obtained from surface mapping, test pits and trenches below the depth limitations of these methods and to:

- provide control for the interpretation of any geophysical investigations.
- provide samples from these greater depths.
- provide access for test equipment e.g. for measurement of water levels, pore pressures and permeability etc.

The major advantage of drilling is that, subject to the choice of a suitable rig, there is little restriction to the depth to which the investigation can be taken. In the assessment of the subsurface profile the properties of the lowest rock or soil unit which could affect the structure usually have to be determined. In many cases drilling provides the most practical method.

Drilling also has little effect on the environment. Maximum hole size is usually less than 200 mm and holes can be easily covered, backfilled or neatly preserved. Surface disruption is commonly restricted to the preparation of a drilling pad on sloping ground.

In site investigation drilling it is imperative to recover as much information as possible on the subsurface profile. The drilling and sampling method should be selected to maximise the recovery of low strength material. Zones of crushed rock, gouge or clay seams are particularly important but some core losses can hardly be avoided. The recording of drilling parameters e.g. the drill penetration rate, drill fluid pressure, can assist in the interpretation of subsurface conditions. Wherever practicable, drill holes should be used for groundwater monitoring for as long as they can be.

5.9.2 Drilling techniques and their application

Table 5.3 summarizes the different drilling techniques available, their applicability, advantages and disadvantages.

In the selection of the drilling method care should be taken not to disturb the existing site conditions. In the investigation of landslides the use of drilling fluid could lead to increased pore pressures, softening and hydraulic fracture. For the same reason

Table 5.3 Drilling techniques and their applicability.

Drilling techniques	*Hole diameter*	*Support*	*Applicability*	*Advantages*	*Disadvantages and precautions*	*Approx. cost & productivity (2014) $Australian*
Auger drilling Solid flight	Usually 100 mm	Self-support i.e. the hole must support itself	Clayey soils, moist silty sand and sand above the water table	Ease of setting up, rapid drilling, continuous disturbed sample recovered	Limited depth – 30 m normal maximum. Not applicable for sandy soils below water table or soft clays because hole needs support	$55–75/m, 30 m/day with stopping for sampling
Auger drilling Hollow flight	Usually 100 or 150 mm, up to 300 mm	Self-support above water table if augers used as for solid flight augers. Augers used to support below water table	As above, and for sandy soils and soft clays below the water table by using the augers for support and sampling through centre of augers	As above, can be used below water table if flights filled with water or mud. Easy sampling. Can be used in dam cores	Hollow flights are not normally filled with water or mud so 'blowing' or disturbance at base of hole may occur in granular soils and soft clays when withdrawing the plug in the base of the augers for sampling or testing	$70–80/m, 20 m to 30 m/day for 100 mm diameter.
Rotary drilling – Non coring in soil & rock (wash boring)	75–100 mm common	Self-support in clayey soils. Casing and/or drilling mud in granular soils and soft clays. Usually no support in rock, but need mud if rock is weak or highly fractured	All soil and rock types	Allows full range of sampling and testing techniques. Rapid drilling in rock.	Poor identification of soil types in drill cuttings. Virtually impossible to identify thin beds of sand in clay. Proper identification requires tube or SPT samples. Unable to penetrate gravels efficiently. Must case or fill hole with water or mud to prevent blowing in loose granular soils and disturbance of soft clays. Note that even stiff clays can fail by shear at 30 m depth if not supported by mud. Rock samples difficult to identify in cuttings. May induce hydraulic fracture in dam cores and should not be used for that purpose	$55–70/m soil, $70–80/m rock plus $50/m for river gravel. Good investigation rotary drill rig with hydraulic top drive and 3 m stroke. Variable speed head for augering/coring. Truck mounted with 2 or 3 winches to facilitate rapid removal of drill rods for testing. Hire rate is approx. $200–250/hr

Rotary drilling. Coring in soil & rock	'N' hole 76 mm, 'H' hole 96 mm. Use 'H' in soil and poor rock	Hole is supported by casing through soil overlying rock. No support in rock. Water, mud or foam used as drill fluid	All rock types. With care good recovery can be achieved in soils with any cohesion	Continuous core of rock. Continuous 'undisturbed' sample allows identification of stratification in alluvial soils	Poor recovery in weak, friable, and closely jointed rock. No recovery in clean sand and silt. Soil and weathered rock samples usually disturbed by stress relief and swell due to water used from drilling. 'H' size core gives better recovery but is heavier and more expensive but the incremental cost is small and usually worth spending	NMLC or NQ3 coring $125–150/m, HMLC or HQ3 coring $140–160/m. 10–15 m/day coring. To ensure best practicable core recovery drilling on hourly hire is recommended. Rig hire $220–290/hr. Add $10/m for bit wear
Reverse circulation (rotary)	Usually BQ Duo-tube	Outer casing of reverse equipment	Sands, silty sand, sand with fine gravel	Rapid drilling. Complete recovery of soil drilled (disturbed). Useful for investigation of sand sources	Less commonly available. More expensive equipment. Mixes strata. No testing or undisturbed sampling possible	Rig hire $200–250/hr plus compressor hire. Up to 30 m/hr
Sonic drilling	75 mm to 250 mm. Commonly available 150 mm.	Outer casing of sonic equipment	All soils and low strength rock. Can penetrate stronger rock but not the preferred method. Can drill up to 150 m with large drill rigs	Rapid drilling. Complete recovery of a "core" of the soil or rock if carried out properly. This includes non-plastic soils such as sands, silts and sand gravel cobble soils. Does not require mud support and only limited water so may be used where hydraulic fracture is a problem such as in dam cores.	Less commonly available. More expensive than conventional rotary drilling, but this may be offset by higher productivity and better sample recovery. The "core" is disturbed by the drilling and by poor handling procedures. This can be reduced by using a plastic inner liner in the core barrel. Potential for sonic vibrations to cause densification of loose sands below the drill bit and adversely affect SPT N values for liquefaction assessments	Rig hire $ 5,500/day.
Cable tool percussion bit and bailer	Up to 750 mm	Casing which may or may not be recovered	Sands, gravel, clay, rock (fractured)	Can penetrate gravels. Can drill large diameter hole. Simple equipment	Slow drilling. Results in strata mixing, breakage of rock fragments and loss of fines which can give misleading particle size distribution. Very heavy tools for transport and handling. Seldom used for site investigations as other methods are better	$120–150/hr. 2 m/hr down to 0.10 m/hr

(*Continued*)

Table 5.3 Continued.

Drilling techniques	*Hole diameter*	*Support*	*Applicability*	*Advantages*	*Disadvantages and precautions*	*Approx. cost & productivity (2014) $Australian*
Cable tool tube	Up to 200 mm	Casing usually recovered	All soils and weak rocks	Provides continuous samples and useful groundwater data	Slow drilling. Very heavy tools for transport and handling. Seldom used for site investigations as other methods are better	Rig Hire $120–150/hr. 2 m/hr down to 0.10 m/hr
Hammer drill (impact drill)	Usually 50–100 mm	Self supporting in rock. Needs casing in soils	Gravel, rockfill, boulders, rock (to just drill a hole)	Very rapid penetration rate (e.g. 200 m in 24 hours). Can penetrate rockfill and soil with gravel and cobbles	Samples limited to rock chips. May jam and lose equipment in loose fractured rock, rockfill or boulders. Penetration in clay is limited due to blockage of air passages	$220–290/hr plus compressor hire
Bucket auger, casing and auger, casing and clam shell	900–1200 mm	Casing which may or may not be recovered	Sands and gravels but applicable to al soils.	Drills through gravely soils without breaking up into smaller sizes. Suitable for investigation of sand/gravel filter sources	Costly. Sub-sampling and testing also expensive because of large sample size	$300/m for holes to 10 m. 10 m/day
Rotary drilling with rig on all-terrain vehicle	As for other rotary drilling	As for other rotary drilling	Drilling in swamps, water up to 0.5 m deep	Allows access where truck mounted rig would not enter because of soft ground	Smaller rig mounted. Applicable to 30 m depth	Bombardier $350 per day in addition to ordinary drilling cost
Rotary drilling with rig on small jack-up barge	As for other rotary drilling	As for other rotary drilling	Drilling in water up to 6 m deep	Float rig into position and set up on bottom, unaffected by tide or waves	Costly to establish. Smaller rig mounted. Applicable to 30 m depth	$3,000–4,000 per day in addition to ordinary drilling cost. May need hire of work-boat to service rig

Other costs: U50 tubes including sampling $60; U75 tubes including sampling $80; SPT $50 each to 30 m; Core boxes $50/box; Water cart $250/day; Compressor $500–600/day; Installation of rotary drill casing 10/m; establishment costs (and accommodation if applicable).

the investigation of the clay core of existing embankment dams should always use dry drilling techniques, e.g. hollow-flight-auger, auger, sonic drilling or dry cable tool.

Drilling has the disadvantage that information obtained is almost always indirect – either from the observation of resistance to rig penetration, by the measurement of *in situ* properties with equipment lowered down the hole or by the logging of samples recovered by the drilling (e.g. Thin Wall Tube samples). Direct observation of the ground is restricted to the use of mirrors, down-the-hole camera, television or fiberscope. Large diameter holes drilled by augers or the calyx method can, in some cases, be entered and logged.

Drilling rates depend on the machine and the material type and typically are 4 to 5 auger holes to 10 m in soil per 8-hour shift or 15 m of core drilling and water pressure testing per 8-hour shift. Rigs are often truck mounted – which makes them mobile but restricts access in sloping ground. Many drill companies now have track-mounted rigs for use in difficult terrain. Skid mounted rigs were relatively common in the past but require considerably more time for establishment at each site.

It is not good practice to specify drilling programmes in detail before geotechnical mapping and logging of test pits and/or trenches is well advanced or completed. By this time, understanding of the geological situation should be sufficient to allow boreholes to be aimed at answering specific geological and/or engineering questions.

5.9.3 Auger drilling

The most common type of drilling in cohesive soils uses a spiral flight auger to penetrate and remove the material below the surface. The simplest form is the hand auger which is usually restricted by the physical effort involved to about 3 m.

Most augers are machine driven – and range from portable to large truck-mounted hydraulic drill rigs (Figure 5.13).

The common auger rig equipped with either 100 mm or 150 mm diameter solid or hollow flight augers, can reach up to 30 m in soil strength materials. A steel blade 'V' bit will penetrate most fine-grained soils and extremely weak rocks but usually refuses on coarse gravel or weak strength rock (Table 2.4). A tungsten-carbide 'TC' bit will grind slowly through weak and medium strong rock.

Auger drilling allows the logging of disturbed material collected from the flights during drilling. By removal of the augers it is practical to regularly recover tube samples and carry out *in situ* testing of the material properties.

Auger drilling is suited to the investigation of areas with thick soil deposits which extend beyond the practical limit of pits or trenches. It does not provide the same amount of data on soil structure but can supplement other information. In many cases it is used as a rapid method of establishing the depth and general properties of the material overlying rock which will be investigated in more detail by some other method.

A major difficulty in auger drilling in cohesionless soils or soft clays is the stability of the sides of the drill hole particularly below water. Figure 5.14 shows a log prepared from an augered drill hole in sandy soils below the water table, compared with the log of a nearby static cone penetrometer probe. Mixing and collapse of the hole have led



Figure 5.13 Large diameter hollow auger drill rig (courtesy CMEC).

to gross inaccuracies in the auger hole log. Auger drilling in non-cohesive and weak soils is therefore restricted to above the water table.

Hollow flight augers have the advantage of providing support for the hole in these conditions and allow sampling through the augers. However the action of removing the plug at the end of the auger will often loosen and disturb the soil below the auger and hollow flight augers are not recommended for drilling in cohesionless soils, particularly if they are loose, or in weak clays.

The development of large diameter hollow auger drilling has enabled the recovery of continuous core with effectively undisturbed soil samples up to 300 mm in diameter (Figure 5.13 and Figure 5.15). This equipment is used to drill up to 80 m deep in dam cores in the USA.

Hollow flight augers are an acceptable technique for drilling in the core of existing dams because no drilling fluid is used which can cause hydraulic fracture. However the holes must be carefully backfilled in stages with cement/bentonite grout to avoid hydraulic fracture.

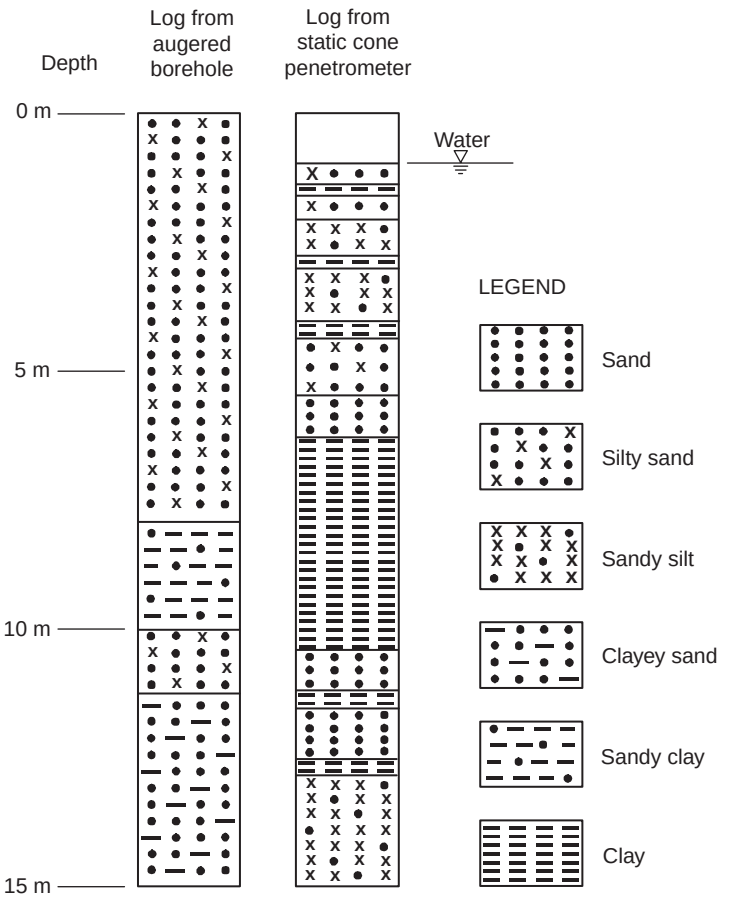


Figure 5.14 Log of sandy and clayey soils below the water table from an augered borehole and from static cone penetrometer.

5.9.4 Percussion drilling

There are three main types of percussion drilling as follows:

a. *Cable tool (bit and bailer) drilling* (Figure 5.16a) involves the successive dropping of a heavy chisel type bit to the bottom of the hole. The fragments are recovered using a bailer. If the hole is dry, water is added to aid the bailing. As the hole is advanced steel casing is driven to preserve the hole. The method is slow, but is effective in penetrating gravels. It should be appreciated that the penetration process involves a reduction in particle size and may mix material from several layers. The grading of samples recovered from cable tool drilling may be different from that of the natural material.

b. *Cable-tool Tube (also called Shell and Auger) drilling*. This method described by Wilson and Hancock (1970) uses a cable-tool rig as in Figure 5.16a, but the hole

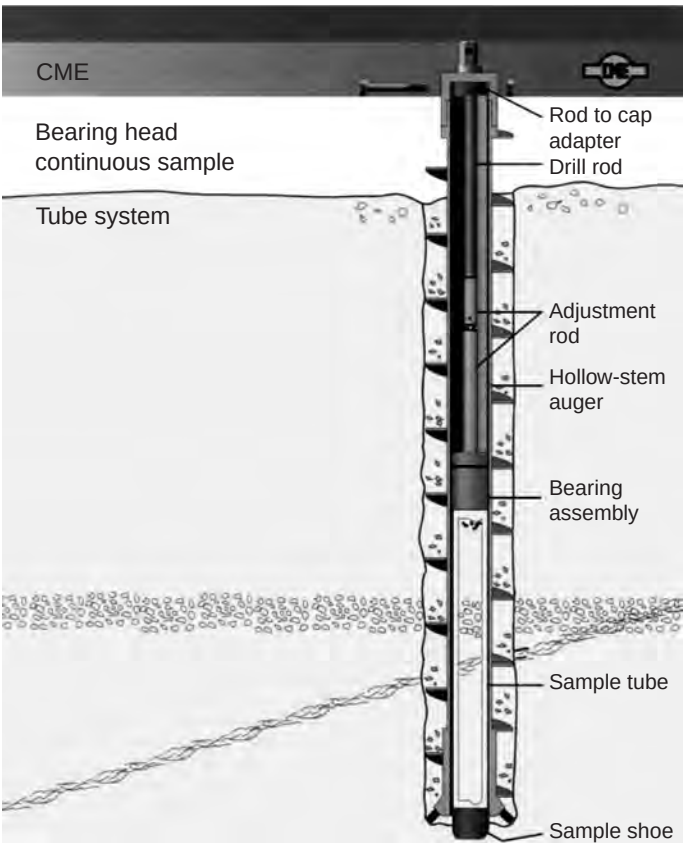
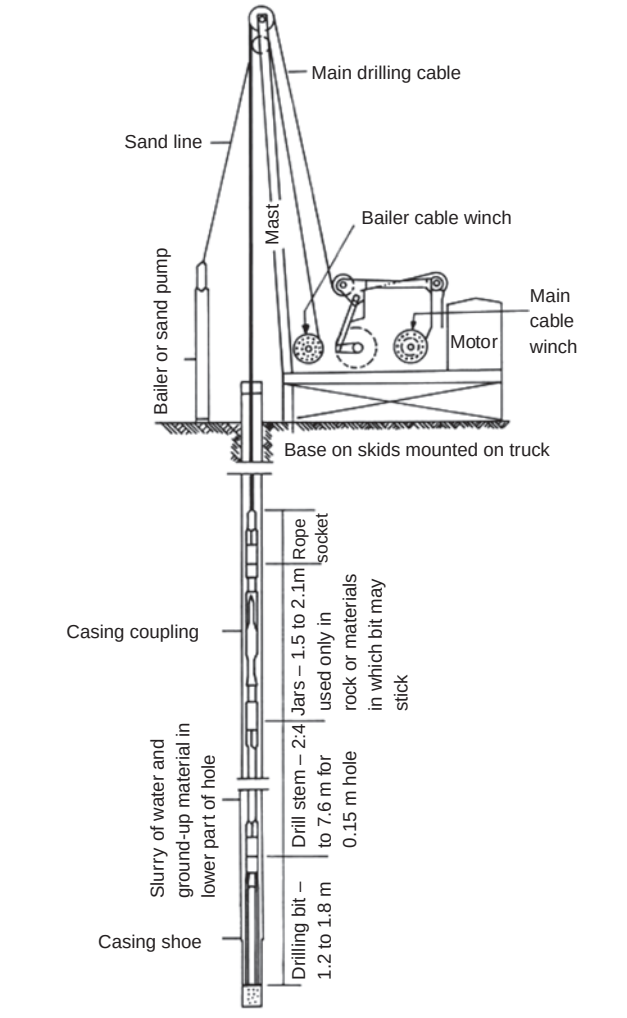


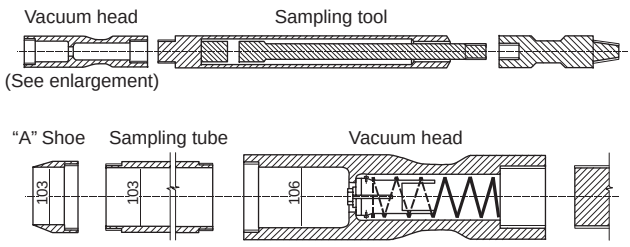
Figure 5.15 Hollow auger continuous sample tube system (courtesy CMEC).

is advanced and sampled by driving 100 mm diameter sampling tubes (Figure 5.16(b) and Figure 5.17). After each 300 mm long tube is driven the sample is extruded into a plastic sleeve, labelled and placed into a core tray. Before the next sample is taken the hole is reamed out using a slightly belled cutting shoe. When required, due to hole collapse or to seal off groundwater flow, the hole is reamed further and casing is driven. No drilling fluid is required. The method provides a continuous geological record in most soil types, a record of moisture content changes and also of the position and source level of any groundwater inflows (Figure 5.17). The tube samples are too disturbed for triaxial or similar testing but the fabric and defects in soils are preserved and can be seen when the samples are split. If rock, coarse gravel or boulders are met, these materials can be penetrated by the bit and bailer method.

Cable-tool drilling can be used for investigating the cores of existing dams as it is a dry method and will not induce hydraulic fracture. However it is slow and expensive compared to auger or sonic drilling. Completion of such holes should include backfilling with grout in stages designed to avoid hydraulic fracture.


Main drilling cable
Sand line
Mast
Bailer cable winch
Bailer or sand pump
Main cable winch
Motor
Base on skids mounted on truck
Rope socket
Jars – 1.5 to 2.1m used only in rock or materials in which bit may stick
Casing coupling
Drill stem – 2:4 to 7.6 m for 0.15 m hole
Slurry of water and ground-up material in lower part of hole
Drilling bit – 1.2 to 1.8 m
Casing shoe

(a) Cable tool rig



(b) Cable-tool sampling equipment

Figure 5.16 (a) Cable tool rig and (b) cable-tool sampling equipment.

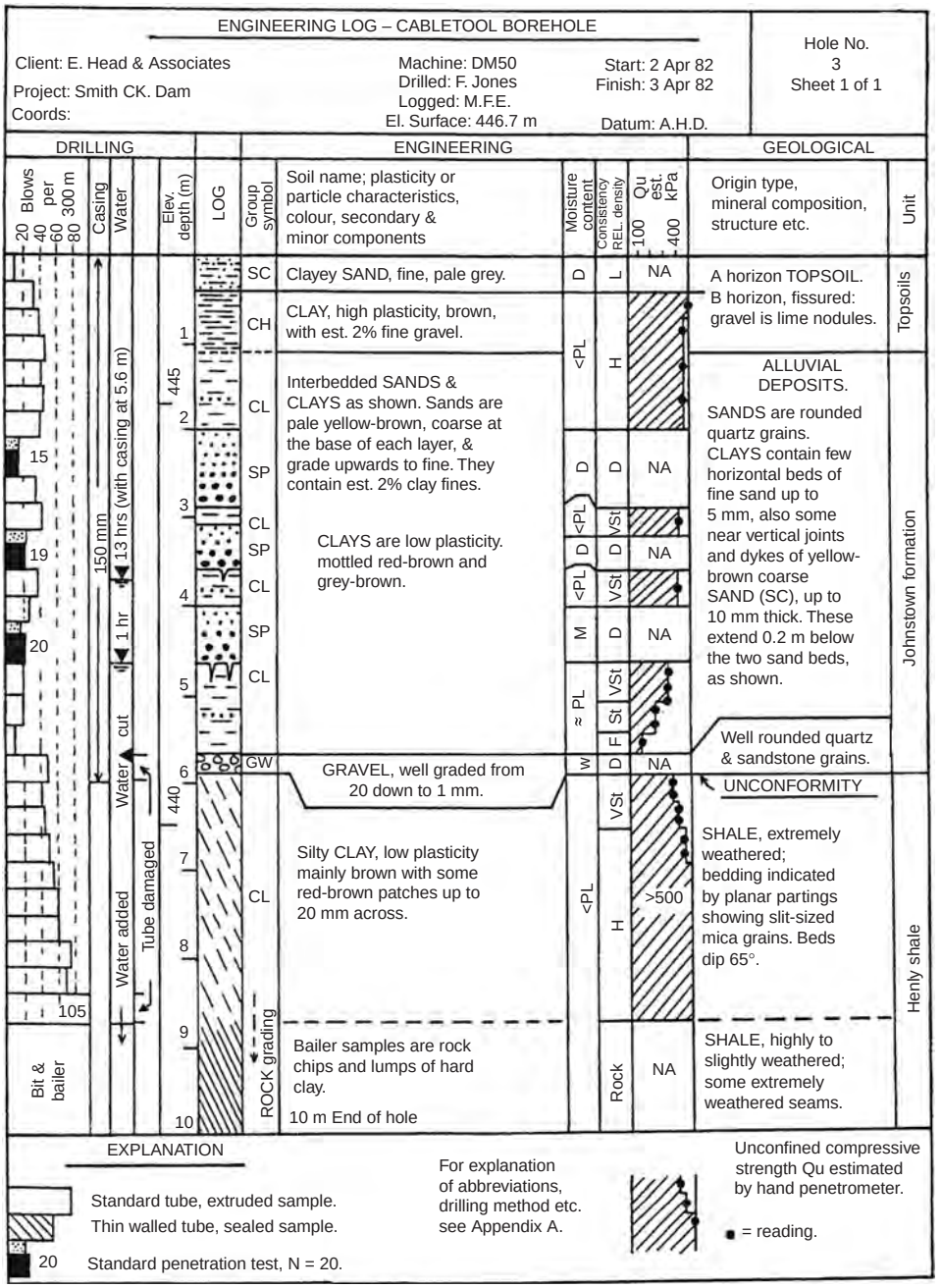

ENGINEERING LOG – CABLETOOL BOREHOLE
Client: E. Head & Associates
Project: Smith CK. Dam
Coords:
Machine: DM50
Drilled: F. Jones
Logged: M.F.E.
El. Surface: 446.7 m
Start: 2 Apr 82
Finish: 3 Apr 82
Datum: A.H.D.
Hole No. 3 Sheet 1 of 1
DRILLING
ENGINEERING
GEOLOGICAL
Blows per 300 m
Casing
Water
Elev. depth (m)
LOG
Group symbol
Soil name; plasticity or particle characteristics, colour, secondary & minor components
Moisture content
Consistency REL. density
Qu est. kPa
Origin type, mineral composition, structure etc.
Unit
SC
Clayey SAND, fine, pale grey.
A horizon TOPSOIL. B horizon, fissured: gravel is lime nodules.
Topsoils
CH
CLAY, high plasticity, brown, with est. 2% fine gravel.
Interbedded SANDS & CLAYS as shown. Sands are pale yellow-brown, coarse at the base of each layer, & grade upwards to fine. They contain est. 2% clay fines.
CLAYS are low plasticity. mottled red-brown and grey-brown.
ALLUVIAL DEPOSITS.
SANDS are rounded quartz grains. CLAYS contain few horizontal beds of fine sand up to 5 mm, also some near vertical joints and dykes of yellow-brown coarse SAND (SC), up to 10 mm thick. These extend 0.2 m below the two sand beds, as shown.
Johnstown formation
GW
GRAVEL, well graded from 20 down to 1 mm.
Well rounded quartz & sandstone grains.
UNCONFORMITY
Silty CLAY, low plasticity mainly brown with some red-brown patches up to 20 mm across.
SHALE, extremely weathered; bedding indicated by planar partings showing slit-sized mica grains. Beds dip 65°.
Henly shale
Bailer samples are rock chips and lumps of hard clay.
10 m End of hole
SHALE, highly to slightly weathered; some extremely weathered seams.
150 mm
13 hrs (with casing at 5.6 m)
1 hr
Water cut
Water added
Tube damaged
ROCK grading
Bit & bailer
EXPLANATION
Standard tube, extruded sample.
Thin walled tube, sealed sample.
Standard penetration test, N = 20.
For explanation of abbreviations, drilling method etc. see Appendix A.
Unconfined compressive strength Qu estimated by hand penetrometer.
= reading.

Figure 5.17 Engineering log of cable tool drill hole (Courtesy South Australian Department of Mines).



Figure 5.18 Rotary drill rig (a) Side view. Mast can be tilted to drill angled holes. (b) Rear view showing top drive hydraulic motor. (c) Dismantled core barrel showing inner tube, core catcher and drill bit (courtesy Jeffery and Katauskas).

c. *Hammer (or Impact) Drilling.* Machines for this type of drilling range from top-hole drill rigs where a pneumatically or hydraulically activated hammer is located at the top of the drill string to large truck-mounted downhole hammers with reverse air or water circulation. These methods provide rapid penetration in medium to very strong rock but the rock quality must be assessed from penetration rate measurements and logging of sand and gravel sized rock fragments

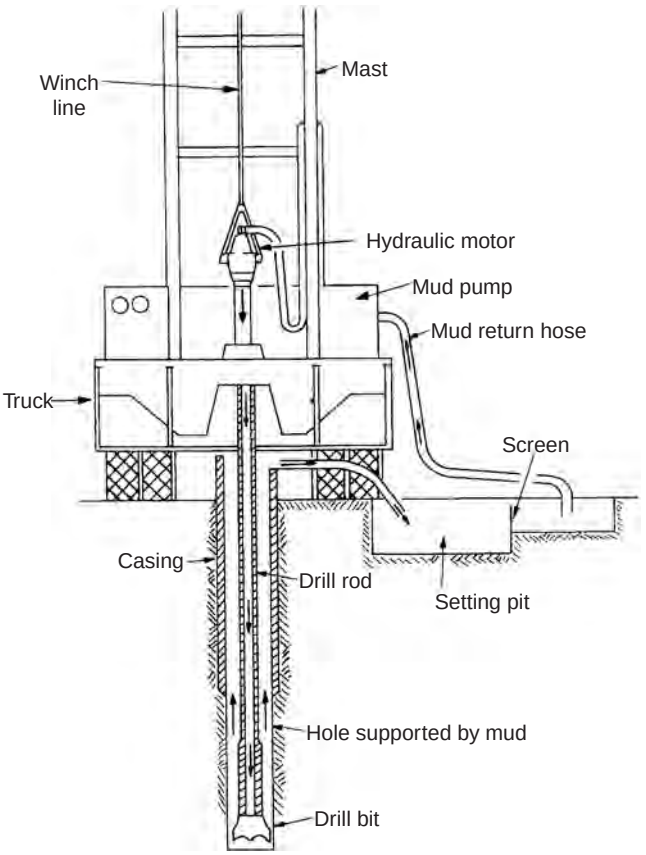


Figure 5.19 Rotary drilling rig-non coring, also known as wash boring.

which provide no data on rock structure. The efficiency of the drilling is affected by excessive groundwater and drilling is slowed if the bit vents become blocked with clay.

It is considered that hammer drilling has little or no application in dam investigations other than through rockfill of existing dams for which it is particularly suited. However if the drilling penetrates into the core of the embankment the very high pneumatic or hydraulic pressures used for drilling are likely to hydro-fracture the core so this drilling must stop short of the core.

Hammer drilling can be useful as a supplement to core drilling in the investigation of overburden depths and rock quality at sites for spillways or quarries.

5.9.5 Rotary drilling

Drilling using a rotary machine (Figure 5.18, Figure 5.19 and Figure 5.20) in soil and rock can involve either core or non-core methods.

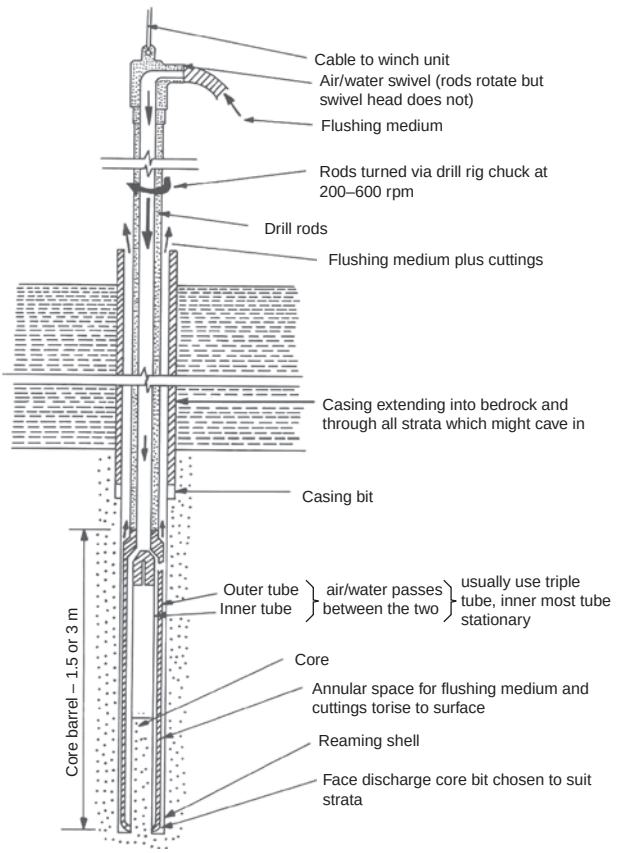

Cable to winch unit
Air/water swivel (rods rotate but swivel head does not)
Flushing medium
Rods turned via drill rig chuck at 200–600 rpm
Drill rods
Flushing medium plus cuttings
Casing extending into bedrock and through all strata which might cave in
Casing bit
Outer tube
Inner tube
air/water passes between the two
usually use triple tube, inner most tube stationary
Core
Annular space for flushing medium and cuttings torise to surface
Reaming shell
Face discharge core bit chosen to suit strata
Core barrel – 1.5 or 3 m

Figure 5.20 Rotary drilling rig-coring.

a. *Non-core drilling*, sometimes called wash boring, involves the use of a solid roller, button or drag bit at the end of drill rods. The bits break up or grind the full face of the bottom of the hole. The soil and rock fragments are removed by the circulating fluid – commonly water. Samples from non-core rotary drilling through soil and rock are usually slurry mixtures of water and fine-grained material. This leads to poor identification of the materials.

It is usual to collect thin-walled tube samples of cohesive soils for more accurate material description and laboratory testing and to carry out *in situ* testing in the borehole.

Non-coring bits can penetrate rock and care must be taken to change to coring methods as soon as resistance to penetration is encountered.

When drilling in sandy soils above the water table and in most soils below the water table, drilling mud is used to support the hole sides and assist in the recovery of drill cuttings. Either bentonite or more commonly chemical mud is used to form a cake on the sides of the hole (Figure 5.21). By maintaining a head

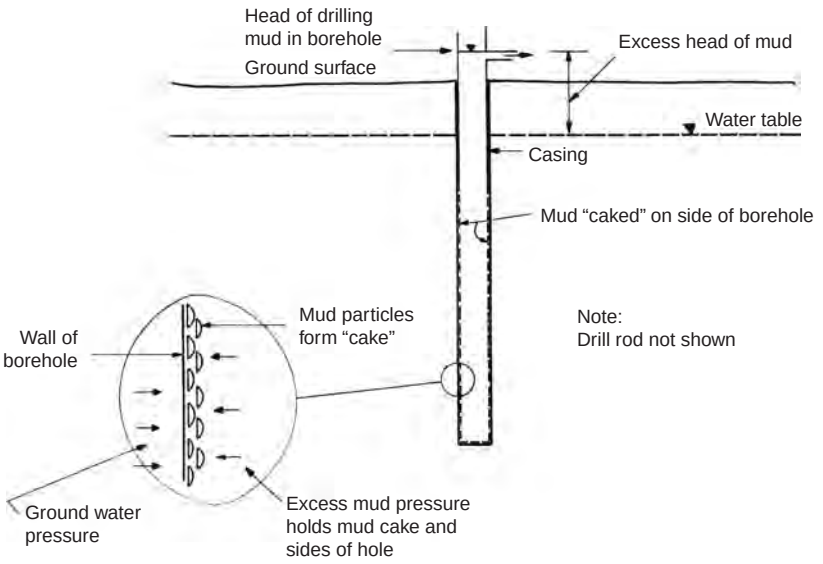


Figure 5.21 Principle involved in the use of drilling mud to support the sides of a borehole.

of mud above the water table the excess pressure supports the sides of the hole. Bentonite mud must remain dispersed to be efficient. In saline groundwater the bentonite may flocculate and reduce the ability of the mud to support the sides of the hole.

Some chemical muds can be designed to maintain their viscous nature for a limited time and revert to the viscosity of water after a few hours. These muds have the advantage of not affecting the permeability of the material surrounding the hole and are suited to holes where permeability testing is required.

Despite the 'reverting' nature of the mud, the authors' experience is that permeabilities obtained by water pressure testing in holes drilled with mud are lower than those obtained in holes drilled with water.

Chemical drilling muds can be used to improve recovery when coring in weak, erodible rock or stiff to hard soils. Core recovery can also be improved by using chemical foam instead of water or mud. This is described in Brand and Phillipson (1984), and illustrated by Figure 5.22.

b. Core Drilling – an annulus of rock is removed using a hollow bit with a leading edge impregnated with fragments of diamond or tungsten carbide, leaving a cylinder, or core, of rock which can be removed by the core barrel. In conventional core drilling (Figure 5.18) the barrel is connected to the end of drilling rods which convey the circulating fluid to the bit. Each time the core barrel is filled the rods have to be removed.

In wireline drilling (Figure 5.23) the use of thin-walled rods and a special core barrel enables the core to be raised inside the rods and significantly reduces the time involved in drilling deep holes. The core recovered by wireline is slightly smaller in diameter than for conventional drilling (NMLC core 52 mm, NQ3 core 45 mm).

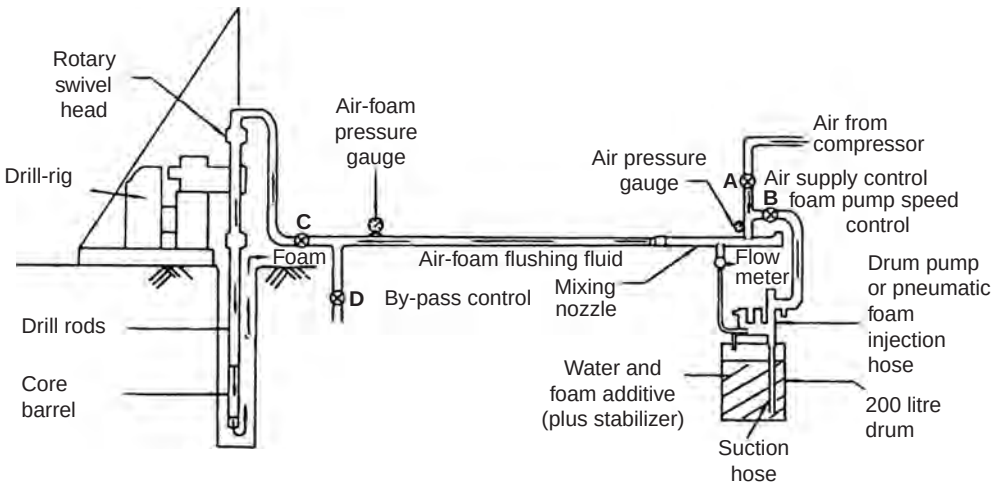


Figure 5.22 Schematic layout of air-foam and flushing system (Brand and Phillipson, 1984).

The objective of site investigation drilling is to obtain the maximum amount of information on subsurface conditions. Every effort is required to recover as much core as possible. In low strength, fractured rock this may involve many short runs with low thrust and drilling water pressures. Triple tube, stationary inner tube, core barrels reduce core disturbance, improve core recovery and should be used in all dam investigations. The authors still occasionally see projects with very poor core recovered with double tube drilling methods. An experienced and quality drill operator is also important.

In the analysis of foundation conditions, particularly in relation to slope movement and potential leakage, the nature of the low strength material is most important. A drilling programme which recovers all the strong rock and loses all the weak material is ineffective and its results may be misleading. As a matter of course drillers should be required to comment on the probable reasons for every section where core has not been recovered.

Drilling contracts should be worded to ensure effective core recovery rather than rapid drilling progress. The loss of 1 m of core in the upper part of the hole may be much more important than the recovery of 10 m of fresh core at the base of the hole

Placement of the core in boxes and logging of the core should emphasise sections where core was not recovered as these may represent crucial zones in the foundation.

The choice of the core size to be used has to be considered in relation to the ability to provide full core recovery. Larger diameter core is somewhat more expensive to drill, but is likely to produce a higher core recovery in low strength and fractured rock.

The ratio between natural fragment size in the rock to be sampled and core diameter is important. It is almost impossible to core conglomerate with 50 mm pebbles with N size equipment (76 mm hole, 52 mm NMLC core, or 45 mm NQ3 core). As a

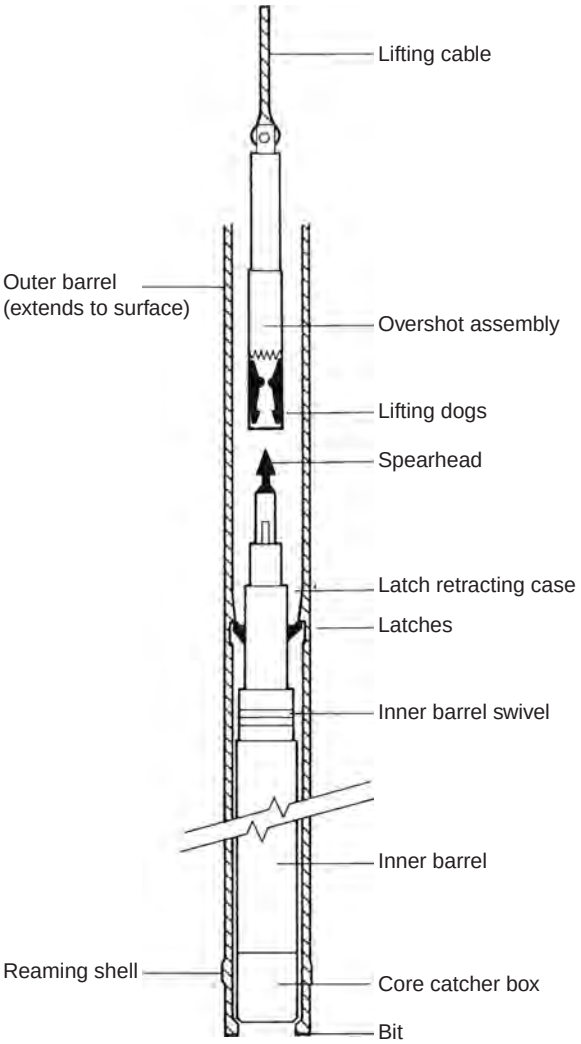


Figure 5.23 Principal components of a wireline drilling system (Clayton *et al.*, 1995).

general rule coarse-grained fractured rock requires large diameter core. N size drilling is commonly used for high-strength rocks, but H size (96 mm hole, 63 mm HMLC core, or 61 mm HQ3 core) is significantly more effective in lower strength materials and only costs about 10% more than N size. PQ size (85 mm core) is more effective again.

On many dam sites defects which control strength and permeability in the dam foundation are close to vertical. Vertical drill holes are unsuitable for sampling these features and in the assessment of their effect on the foundation permeability.

The ability to drill angled holes at selected azimuths enables the choice of hole direction to provide maximum information including the orientation of rock defects and to interact defects which control the rock mass permeability. Angled holes are commonly slower to drill and slightly more expensive (contract rates are usually 10% to 15% higher than for vertical drilling). Drill holes at angles of less than 45° to the horizontal require anchoring of the rig to provide thrust.

5.9.6 Sonic drilling

Sonic drilling has become the drilling method of choice for some organisations for drilling in the core of embankment dams and for some foundation drilling.

The method utilizes a dual casing system, the inner being the core barrel, the outer casing to support the hole. The drill uses intense (50–150 Hz) mechanical vibration from the hydraulically driven oscillator mounted above the casing as shown in Figure 5.24, as well as rotation and downward pressure. The vibrations coincide with the natural resonate frequency of the drill pipe.

As shown in Figure 5.25 the drill method involves advancing the core barrel the full stroke of the rig (usually 1.5 m or 3 m), then the outer casing is advanced to the same depth and the core barrel is withdrawn to the surface and the core extruded by vibrating the barrel while applying water pressure.

The core is collected in clear plastic flexible tube. The core is usually 100 mm or 150 mm diameter. A stiff plastic inner tube can be used for 100 mm dia. core and this reduces the amount of disturbance of the core.

The method usually gives very high penetration rates and can be used for drilling up to 160 m depths in plastic and non plastic soils including soils with gravel and cobble sizes. For deeper holes telescoping casing may be used to reduce the skin friction so only smaller core can be obtained at depth.

Sonic drilling can be used to drill into rock and may be useful for that where the rock is weathered and difficult to core using conventional rotary coring methods. However it usually results in more disturbed core and if the drillers are not experienced just pulverized rock is recovered.

The method has gained acceptance as a 'dry' drilling method in the core of dams so avoiding the potential for hydraulic fracture. In practice the drillers prefer to use some water to lubricate the inner barrel during advancement and to cool the casing and core barrel bits. This gives higher rates of advancement and better core quality.

However as discussed in Stare *et al.* (2013) great care must be taken or high water pressures and hydraulic fracture may occur. They recommend that if water is used it only be used in very limited and quantified amounts. Ideally it should not be used when advancing the inner casing as it results in disturbance of the upper part of the core. If used the inner casing must unobstructed after removing every sample (material often travels up a length of the sampler and into the overlying drill rods). The casing must be open to the atmosphere to prevent the downward advance of the casing pressurizing the water in the hole.

The core obtained by sonic drilling is disturbed. In plastic soils such as in a core of an embankment the amount of disturbance is not great particularly if the inner lining is used. The outer skin of the core is completely disturbed and smears the surface. In non-plastic soils the layering and fabric of the soil can be seen if the drilling is carried out

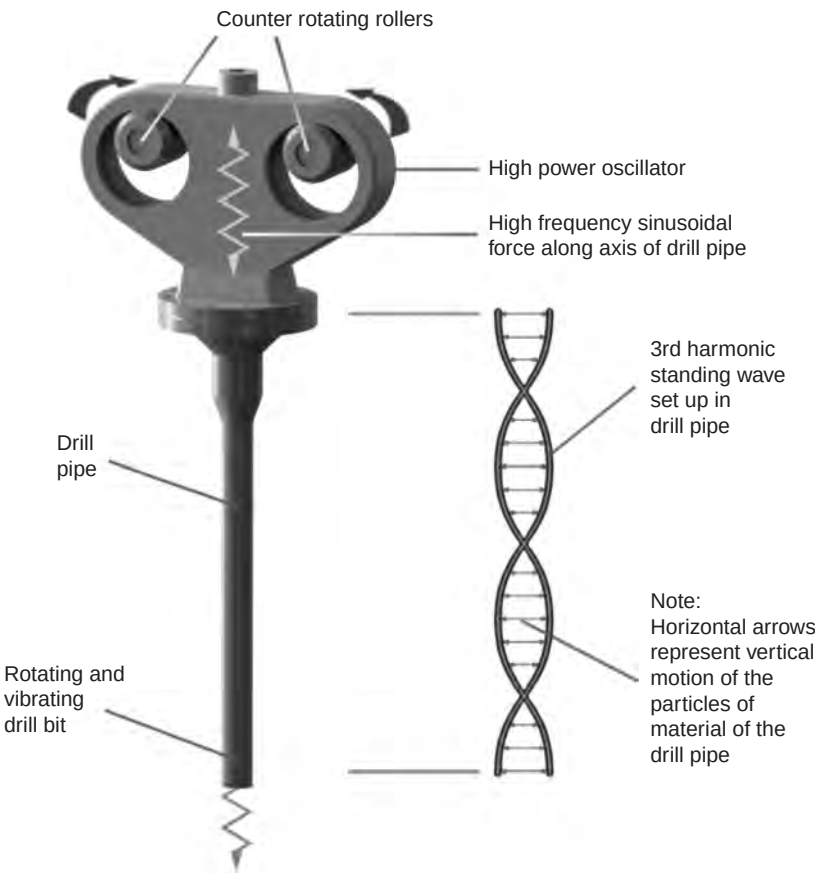


Figure 5.24 Principles of the sonic vibration action (Sonic Drill Corporation).

carefully but the sample will commonly be extended during the handling process. It can still be useful in such materials for investigating of the filters and transitions in existing dams, for investigating alluvial foundations and for determining the stratigraphy and sampling of alluvial foundations subject to liquefaction.

It is possible to take thin wall tubes and to carry out Standard Penetration Tests in sonic drill holes. The process is to stop advancing by sonic method, withdraw the core barrel and lower thin wall tube or SPT down the outer casing. In non-plastic soils there is a potential of the base of the hole "blowing" or loosening as the core barrel is removed. This can be countered by filling the casing with drill mud as the barrel is removed. However this may cause problems of potential hydraulic fracture in some situations. There is also a possibility that the sonic drilling action will densify the soil below the base of the hole and the SPT will not be properly representative of the properties of the soil. The authors prefer to use the sonic drillingmethod to determine the stratigraphy and allow sampling of the soils and do SPT in separate holes drilled using rotary drilling with mud support.

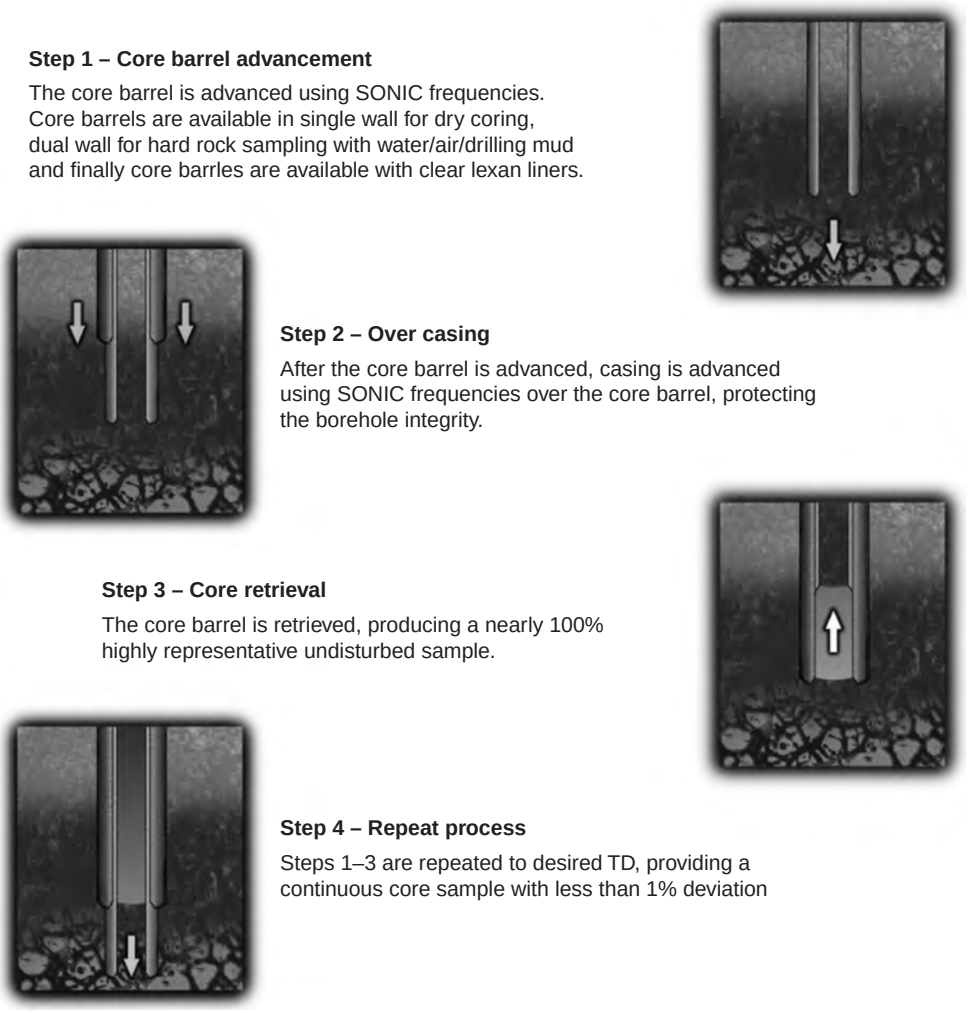

Step 1 – Core barrel advancement
The core barrel is advanced using SONIC frequencies. Core barrels are available in single wall for dry coring, dual wall for hard rock sampling with water/air/drilling mud and finally core barrles are available with clear lexan liners.
Step 2 – Over casing
After the core barrel is advanced, casing is advanced using SONIC frequencies over the core barrel, protecting the borehole integrity.
Step 3 – Core retrieval
The core barrel is retrieved, producing a nearly 100% highly representative undisturbed sample.
Step 4 – Repeat process
Steps 1–3 are repeated to desired TD, providing a continuous core sample with less than 1% deviation

Figure 5.25 Steps in the sonic drilling process (Sonic Drill Corporation).

In some soils the core from a sonic drill hole can "plug" inside the core barrel. This can be detected by comparing the length of core to the length of hole advancement. In other cases where drilling in well compacted clay fill in dams, the length of core recovered will be greater than the length advanced, by up to 40% in the most extreme cases. This occurs because sonic drilling is a displacement method with no circulation of the drill cuttings. These have to be forced into the soil surrounding the core barrel and if the soil is well compacted the path of least resistance is to force the cuttings into the core instead.

In both cases the effects can be reduced by using shorter drill runs, say 0.4 m rather than 1.0 m.

5.10 SAMPLING

5.10.1 Soil samples

Soil samples recovered from the subsurface site investigation are described as either disturbed or undisturbed.

Disturbed samples are collected from pits, trenches and from auger flights as representative of different material types or units. They are identified by location and depth and stored in sealed containers (usually plastic bags) for laboratory testing. For cohesive soils about 3 kg of sample is required for classification testing and 30 kg for compaction tests to evaluate probable performance as engineered fill in an embankment.

Undisturbed samples consist of material which is extracted from the site and transported to the laboratory with a minimum of disturbance. The ideal sample is a cube of approximately 0.3 m sides, hand cut from a test pit, carefully packed and sealed on site and transported to the laboratory without delay.

It is more usual, due to economic factors, and the limitations of depth of test pits, to use thin-walled steel tubes ('Shelby' tubes) to obtain samples of cohesive soils from boreholes. The tubes are pushed into the soil using an adaptor connected to the drill rods. Care should be taken to ensure that the drill hole is cleaned out before sampling and that the drilling water or mud level is maintained during sampling to avoid 'blowing' of material into the hole. Choice of sample diameter depends on the hole size but, in general, the larger the sample diameter the less the disturbance. Common thin walled tube sizes are 50 mm, 63 mm and 75 mm. The wall thickness and cutting edge shape are defined by codes to limit sample disturbance.

Samples should be identified and sealed against moisture loss using either a sample tube sealing device or several layers of molten wax as soon as the sample is recovered from the hole. Before sealing, loose/disturbed soil at the top of the tube should be removed, so moisture does not migrate from that area to the less disturbed part of the sample. Undisturbed samples should preferably be tested within two weeks of sampling as they rust into the tube, or despite all efforts, dry out. On extrusion in the laboratory a proportion of 'undisturbed' samples often prove to have been partly disturbed by the sampling process and it is prudent to take enough samples to allow for this.

In any case, even apparently undisturbed samples are affected by stress changes during sampling, and this needs to be recognised when analysing the results.

5.10.2 Rock samples

Samples of rock exposed in the sides/floor of pits/trenches can be taken for testing of substance strength and mineralogy. Samples should be individually numbered and located. Storage in plastic bags prevents loss of field moisture.

The most common form of rock sample is core. A drill hole with full core recovery should present a complete linear profile through the rock mass below the ground surface. It is important that the core is systematically stored, properly logged, photographed and sampled as soon as practicable after drilling.

In most cases the core needs to be kept for the following purposes:

- To enable the site investigator to make an accurate, clear and concise log of those characteristics of the core which are significant to the project and to use this log in the compilation of the geotechnical model.
- To provide samples for testing.
- For inspection by designers and those preparing bids to build the dam.
- For inspection by those reviewing dam safety during the operating life of the dam. To allow this, core should be stored indefinitely.

The characteristics of most significance to the project will usually be:

- the substance strength
- the lengths of the individual pieces of core
- the lengths and positions of core losses
- the length, position and engineering character of important defects such as sheared or crushed zones
- the engineering character of joint surfaces
- the depths at which significant changes in rock substance type or strength occur.

The systematic method of boxing core is illustrated by Figure 5.26 and Table 5.4. This method 'reconstructs' the core in the core box, to scale, packed with all joint or broken faces fitting, without any spacers apart from sections of core loss. The box is constructed in compartments of convenient lengths – usually either 1 m or 1.5 m – which becomes the 'scale' for use by the driller and others who examine the core.

Changes in moisture content can produce significant changes in substance strength. Representative samples should be sealed, as wrapping in the thin plastic film used for food is only effective in the short term.

Core can deteriorate rapidly on exposure and should be stored inside in covered boxes. This is a particular problem for some shales and siltstones which can slake to soil within several days of sampling.

5.11 *IN SITU* TESTING

In many cases it is preferable to measure the properties of soil and rock in dam foundations using *in situ* tests, rather than taking samples and testing in the laboratory. In some cases (e.g. estimation of the relative density of sands) *in situ* testing is the only method available.

The topic is wide and rapidly developing and this section is restricted to directing readers to the available literature.

5.11.1 *In situ* testing in soils

The most commonly used *in situ* tests in soils are:

- Standard penetration test
- Static cone penetrometer and piezocone

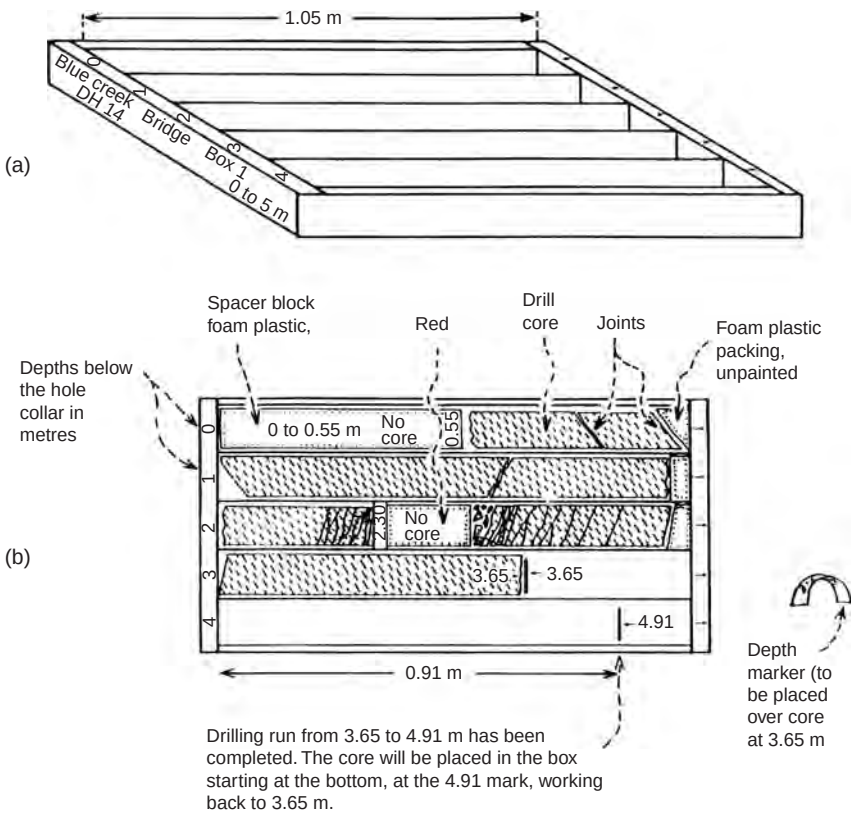


Figure 5.26 Systematic boxing of drill core. (a) Empty core box labelled ready for use; (b) Partly filled box during drilling.

- Vane shear
- Pressuremeter
- Dilatometer
- Plate bearing test

Jamiolkowski *et al.* (1985) and Wroth (1984) give overviews of the topic, and conference proceedings, e.g. Penetration Testing 1988, ISOPT-1 (de Ruiter, 1988), and Pressuremeter Testing (ICE, 1989) contain state of the art papers on the individual methods.

The main methods and their applicability to dam engineering are:

a) *Standard penetration test (SPT)*. Refer to Decourt *et al.* (1988), Skempton (1986) and Nixon (1982) for details.

The test is useful to obtain estimates of the relative density (density index), effective friction angle and deformation modulus (E) of cohesionless soils, and

Table 5.4 Good practice in boxing drill core.

Item	*Description*
Tray length	1.05 m or 1.10 m internal to hold 1.0 m of core and allow for increase in length due to bulking or oblique breakage.
Boxing scale	Core is boxed to scale in the ground; each row in the box represents 1 m of hole length. Any core loss is filled by a block of timber or equivalent, painted red, the length of the core loss and positioned where the core is considered to have been lost. If the position is uncertain the block is placed at the top of the run.
Boxing layout	Core is stored in the box like the lines of writing on a book.
Box marking	The left hand external end of the box is identified with the following information: Project, hole no., box no., depths. The same information should be recorded inside the box. The drilled depth in metres from the surface is marked on the left hand end of each row. Detachable core tray lids should not be marked.
Core marking	The bottom end of the core in each run is marked by paint or a 'permanent' felt pen with the depth of the hole and a labelled line at that depth is marked on the bottom of the tray. A depth marker (Figure 5.22) is placed over the core at the bottom of each run with the depth recorded on the marker. Runs should not be separated by spacers. Where the core tends to break along oblique layering, or where core is very broken, the full 1.05 m or 1.10 m may be required to store the 1 m of core drilled. Where the core has few joints and can be broken normal to the axis the extra length allowed for bulking may not be needed and can be filled with white plastic foam to keep the core tightly packed and avoid disturbance.
Core samples	Core samples are removed only by authorised persons and are replaced immediately by yellow blocks of equivalent length marked with the name of the person who removed the core, the date of removal and reason for removal.

to assess the liquefaction potential of saturated sands and silty sands. The test is widely available but non-standardised and inherently approximate.

b) *Static cone penetrometer (CPT) and piezocone (CPTU)*. Refer to De Beer *et al.* (1988), Campanella and Robertson (1988), Jamiolkowski *et al.* (1988) and Kulhawy and Mayne (1989) for details. Figure 5.27 shows details of an electric cone penetrometer.

The tests are useful to obtain estimates of the relative density, effective friction angle, drained Young's modulus (E) of cohesionless soils, and the undrained shear strength of soft cohesive soils. There are many methods of interpretation of cone penetrometer results. The authors consider that the soil classification using a piezocone proposed by Campanella and Robertson (1988) and modified in Robertson (2010a) to use normalized parameters Q_{tn}, F_r, and B_q to allow for overburden stress provides the most reliable interpretation (Figure 5.28). The cone resistance q_t is obtained from the CPTU q_c value adjusting for cone geometry. For CPT, Figure 5.28 can be used using q_c instead of q_t. The friction ratio, (sleeve friction/q_c) × 100%, is an output of CPT and CPTU tests. B_q is obtained from the CPTU dynamic pore pressure response. That graph is only used as a check on the other for CPTU tests.

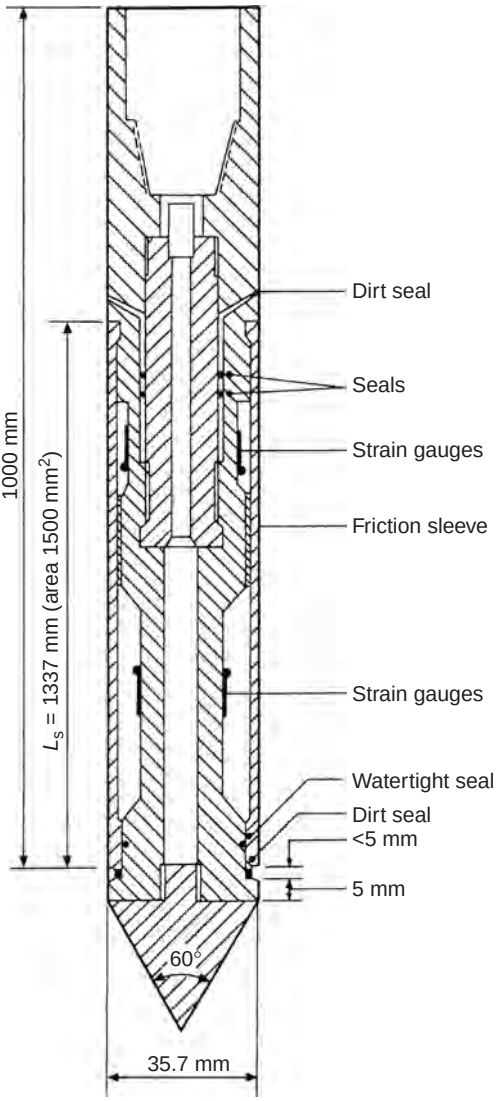
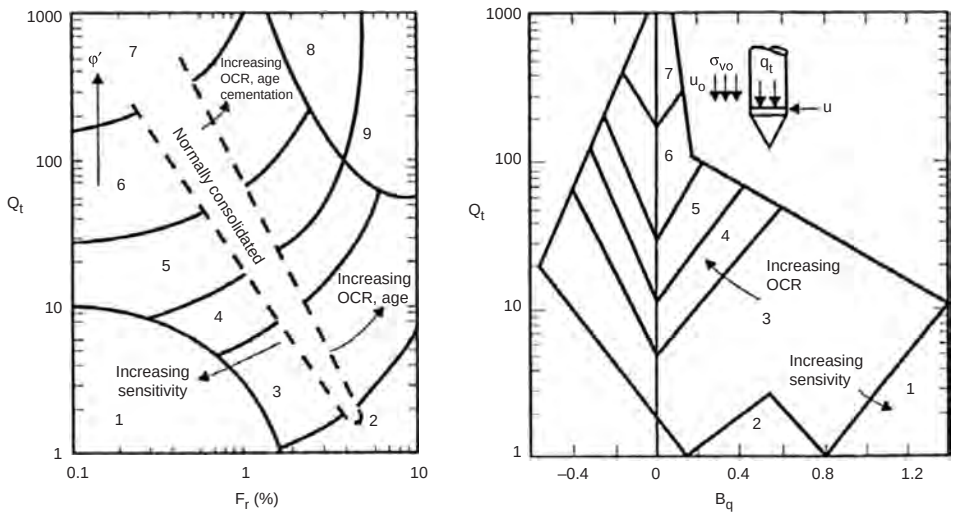


Figure 5.27 Electric cone penetrometer.

The CPT and CPTU are particularly useful in alluvial foundations where sandy soils are inter-layered with clayey soils as the instrument is able to detect the layering better than most drilling and sampling techniques. This test can also be used to assess the liquefaction potential of saturated cohesionless soils. The CPT has been used very successfully to locate softened/weakened zones in earth dams, including the core. The hole formed by the cone should be backfilled with cement/bentonite grout, placed by passing the grout down a tube in the hole under gravity, staged if necessary to avoid hydraulic fracture.



$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \qquad F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

Zone	Soil behaviour type
1.	Sensitive, fine grained;
2.	Organic soils-peats;
3.	Clays-clay to silty clay;
4.	Silt mixtures clayey silt to silty clay
5.	Sand mixtures; silty sand to sand silty
6.	Sands: clean sands to silty sands
7.	Gravelly sand to sand;
8.	Very stiff sand to clayey sand
9.	Very stiff fine grained

Figure 5.28 Normalized CPT Soil Behaviour Type (SBTN) charts using piezocone (Robertson 2010a).

The CPT is widely available but non-standard. The CPTU is somewhat less widely available and requires some corrections to allow for the different designs of equipment. Both are inherently approximate but are generally regarded as more precise than the SPT.

c) *Vane shear*. Refer to Bjerrum (1973), Aas *et al.* (1986), Walker (1983), Azzouz *et al.* (1983) and Wroth (1984) for details.

This test can only measure the undrained shear strength of very soft to firm clays (maximum undrained shear strength of about 70 kPa), and has little use in dam engineering unless there are poorly compacted, puddle clay fill, saturated weak clay fills or the foundations are weak clays. The need for correction of the field vane strength to give design strengths is well documented in Bjerrum (1973), Azzouz *et al.* (1983) and Aas *et al.* (1986).

d) *Pressuremeter*. Refer to Ervin (1983), Jamiolkowski *et al.* (1985), Wroth (1984), Campanella *et al.* (1990), Powell (1990) and Clarke & Smith (1990) for details.

Both the self-boring (SBPM) (Figure 5.29) and Menard type pressuremeters have application in dam engineering, particularly in the assessment of the modulus of soil and weathered rock foundations. These pressuremeters give the most accurate estimate of modulus provided that the test method and interpretation are correct.

The instruments can be used to estimate the effective friction angle of cohesionless soils and the undrained shear strength of cohesive soils. However disturbance

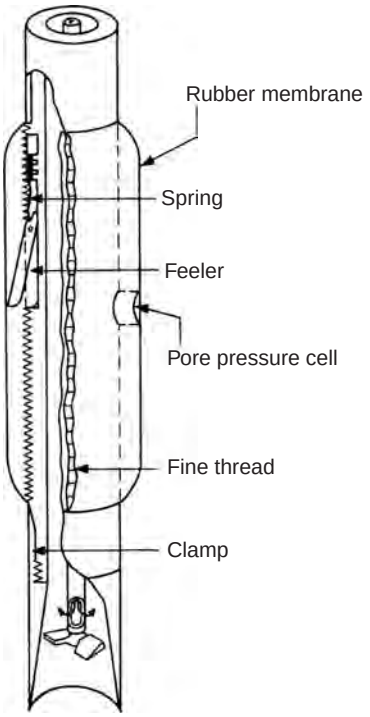


Figure 5.29 Features of the self-boring pressuremeter.

on insertion gives an over-estimate of the undrained strength, so the results should be viewed with caution. SBPM test results can also be used to estimate the *in situ* horizontal stress in the ground, an important factor in modelling deformations in dam foundations using finite element methods and to establish *in situ* stress conditions for dynamic analysis of liquefaction. The tests are relatively expensive (in comparison to CPT and SPT) but affordable on most large dam site investigations.

e) *Dilatometer*. Refer to Kulhawy and Mayne (1989), Lacasse & Lunne (1988). Powell and Uglo (1988) for details.

The (Marchetti) dilatometer can be used in a similar way to the CPT. Its proponents claim advantages over the CPT but these are not apparent to the authors at this time. The dilatometer is far less commonly used in practice.

f) Plate bearing tests. Refer to Pells (1983), and Powell and Quarterman (1988) for details.

Plate bearing tests on the surface, in pits or down hole can be useful in assessing the deformation modulus of soil and weathered rock in a dam foundation. However as the tests are usually restricted to near the ground surface they are of limited value in a situation with a thick compressible layer.

Table 5.5 reproduced from Lunne *et al*. (1997) summarises the applicability of the different *in situ* tests. The authors broadly agree with the table.

Table 5.5 The applicability and usefulness of *in situ* tests (Lunne *et al.*, 1997).

		Soil Parameters												*Ground Type*						
Group	*Device*	*Soil Type*	*Profile*	*u*	$*\phi'$	S_u	I_D	m_v	c_v	k	G_o	σ_h	*OCR*	σ-ε	*Hard Rock*	*Soft rock*	*Gravel*	*Sand*	*Silt*	*Clay*	*Peat*
Penetrometers	Dynamic	C	B	–	C	C	C	–	–	–	C	–	C	–	–	C	B	A	B	B	B
	Mechanical	B	A/B	–	C	C	B	C	–	–	C	C	C	–	–	C	C	A	A	A	A
	Electric (CPT)	B	A	–	C	B	A/B	C	–	–	B	B/C	B	–	–	C	C	A	A	A	A
	Piezocone (CPTU)	A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C	–	C	–	A	A	A	A
	Seismic (SCPT/SCPTU)	A	A	A	B	A/B	A/B	B	A/B	B	A	B	B	B	–	C	–	A	A	A	A
	Flat dilatometer (DMT)	B	A	C	B	B	C	B	–	–	B	B	B	C	C	C	–	A	A	A	A
	Standard penetration test (SPT)	A	B	–	C	C	B	–	–	–	C	–	C	–	–	C	B	A	A	A	A
	Resistivity probe	B	B	–	B	C	A	C	–	–	–	–	–	–	C	C	–	A	A	A	A
Pressuremeters	Pre–bored (PBP)	B	B	–	C	B	C	B	C	–	B	C	C	C	A	A	B	B	B	A	B
	Self boring (SBP)	B	B	A[1]	B	B	B	B	A[1]	B	A[2]	A/B	B	A/B[2]	–	B	–	B	B	A	B
	Full displacement (FDP)	B	B	–	C	B	C	C	C	–	A[2]	C	C	C	–	C	–	B	B	A	A
Others	Vane	B	C	–	–	A	–	–	–	–	–	–	B/C	B	–	–	–	–	–	A	B
	Plate load	C	–	–	C	B	B	B	C	C	A	C	B	B	B	A	B	B	A	A	A
	Screw plate	C	C	–	C	B	B	B	C	C	A	C	B	–	–	–	–	A	A	A	A
	Borehole permeability	C	–	A	–	–	–	–	B	A	–	–	–	–	A	A	A	A	A	A	B
	Hydraulic fracture	–	–	B	–	–	–	–	C	C	–	B	–	–	B	B	–	–	C	A	C
	Crosshole/downhole/ surface seismic	C	C	–	–	–	–	–	–	–	A	–	B	–	A	A	A	A	A	A	A

Applicability: A = high; B = moderate; C = low; – = none.; $*\phi'$ = Will depend on soil type; [1] = Only when pore pressure sensor fitted; [2] = Only when displacement sensor fitted. Soil parameter definitions: u = *in situ* static pore pressure; ϕ' = effective internal friction angle; S_u = undrained shear strength; m_v = constrained modulus; c_v = coefficient of consolidation; k = coefficient of permeability; G_o = shear modulus at small strains; σ_h = horizontal stress; *OCR* = overconsolidation ratio; σ-ε = stress-strain relationship; I_D = density index

5.11.2 *In situ* testing of rock

The most common used *in situ* tests in rock are:

- water pressure permeability test
- pressuremeter
- plate bearing
- borehole orientation
- borehole impression.

The water pressure test is described in Section 5.14. The references given in Section 5.11.1 for pressuremeter and plate bearing tests cover testing in rock as well as in soil.

5.11.2.1 *Borehole orientation*

Refer to Hoek and Bray (1981) and Sullivan *et al.* (1992) for details.

The logging of samples of soil and rock recovered from drill holes can identify the nature and spacing of defects and orientation in relation to the axis of the hole (more commonly expressed with reference to the plane normal to the core axis), but the absolute orientation is required for analysis of the pattern of defects which affect the project. It is the authors' experience that rock mass defects in the drill core can be effectively oriented provided that they can be related to distinctive features (e.g. bedding), which have been identified and oriented on the surface or in underground exposures.

Remanent magnetism can be used to orient drill core from Coal Measure Rocks (Schmidt 1991). This technique involves the establishment of the regional orientation of remanent magnetism. Specific samples can then be oriented in relation to this direction.

It is possible to orient core from an angled drill hole using several different methods.

The simplest method applies when a hole is angled through rock in which the orientation of distinct bedding, foliation or cleavage is known, and is believed to remain constant throughout the depth of the hole. A length of core is simply held in the same orientation as the borehole, and then rotated until the bedding, foliation or cleavage lies in its known orientation. The orientation of joints or other defects can then be measured by compass and clinometer.

Other methods require down-the-hole instruments such as the Craelius core orientation device shown in Figure 5.30.

Analysis of the results of these measurements should consider the inherent problems involved. Accuracy of better than 20° in azimuth is difficult.

5.11.2.2 *Borehole impression packer*

This instrument records defects on the sides of the drill hole (Figure 5.31). It involves a tube with an expandable rubber packer, within foam-covered split metal leaves and a sleeve of thermoplastic film. When the packer is inflated – by water or gas – an impression is retained on the film of the defects which are present on the side of the hole.

If the impression packer is oriented (using marked drill rods), an impression survey should provide orientation data on defects from both vertical and angled drill holes.

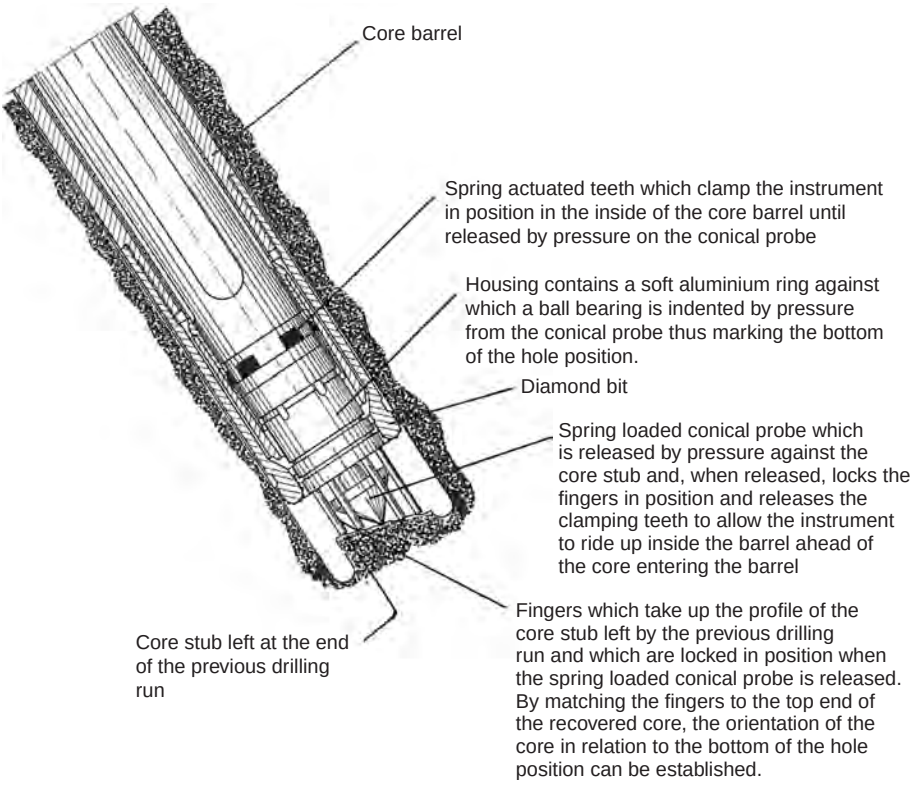

Core barrel
Spring actuated teeth which clamp the instrument in position in the inside of the core barrel until released by pressure on the conical probe
Housing contains a soft aluminium ring against which a ball bearing is indented by pressure from the conical probe thus marking the bottom of the hole position.
Diamond bit
Spring loaded conical probe which is released by pressure against the core stub and, when released, locks the fingers in position and releases the clamping teeth to allow the instrument to ride up inside the barrel ahead of the core entering the barrel
Fingers which take up the profile of the core stub left by the previous drilling run and which are locked in position when the spring loaded conical probe is released. By matching the fingers to the top end of the recovered core, the orientation of the core in relation to the bottom of the hole position can be established.
Core stub left at the end of the previous drilling run

Figure 5.30 Craelius core orientation device (Hoek and Bray 1981).

In practice it has been found that successful use of the impression packer has been limited to medium strong to strong rocks with widely spaced open joints. Closed defects may not form a sufficient indentation in the wall of the drill hole to show on the sleeve. Tearing of the thermoplastic film is common in fractured rock. The method is seldom used now as borehole imaging methods are better and more cost effective.

Analysis of the results of these measurements should consider the inherent problems involved. Accuracy of better than 20° in azimuth is difficult.

5.11.2.3 *Borehole imaging*

Borehole imaging has largely replaced core orientation and bore hole impression devices. There are four types of imaging:

- Optical
- Acoustic
- Electrical
- Combined acoustic and electrical.

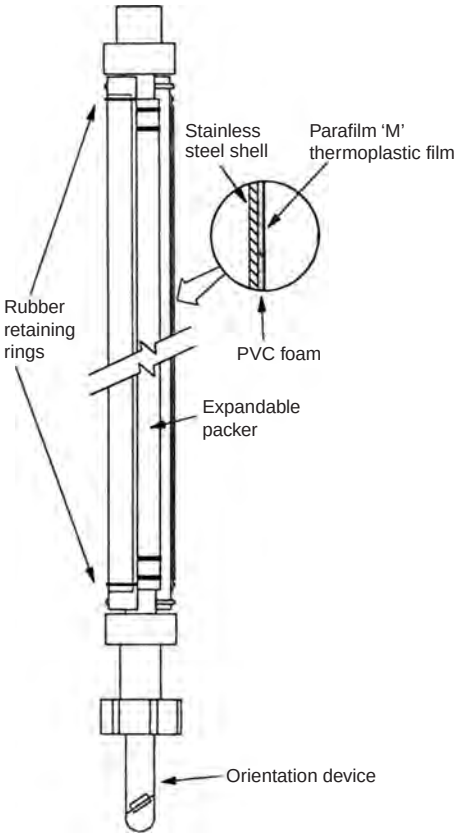


Figure 5.31 Borehole impression packer (Clayton *et al.*, 1995).

Optical imaging is limited in application because it requires a clear fluid in liquid filled holes. It can be readily applied in holes above the water table which are air filled.

Acoustic imaging can be used with mud filled holes. It works by emitting short bursts of acoustic energy which undergo partial reflection at the borehole wall. It can provide a 360 degree image. Figure 5.32 shows an example of an acoustic televiewer log.

The authors experience with these logs is very positive. It allows logging of the stratigraphy, bedding orientation and opening, defect (joints) mapping orientation and opening and allows development of polar diagrams of bedding and joint orientations direct from the log.

It is essential that the interpretation of the image is supported by logging of the drill core.

Electrical imaging uses micro-resistivity. A set of pads and flaps containing an array of button electrodes is used. The authors are not aware of its use in geotechnical investigations.

Gaillot *et al.* (2007) gives details of these methods.

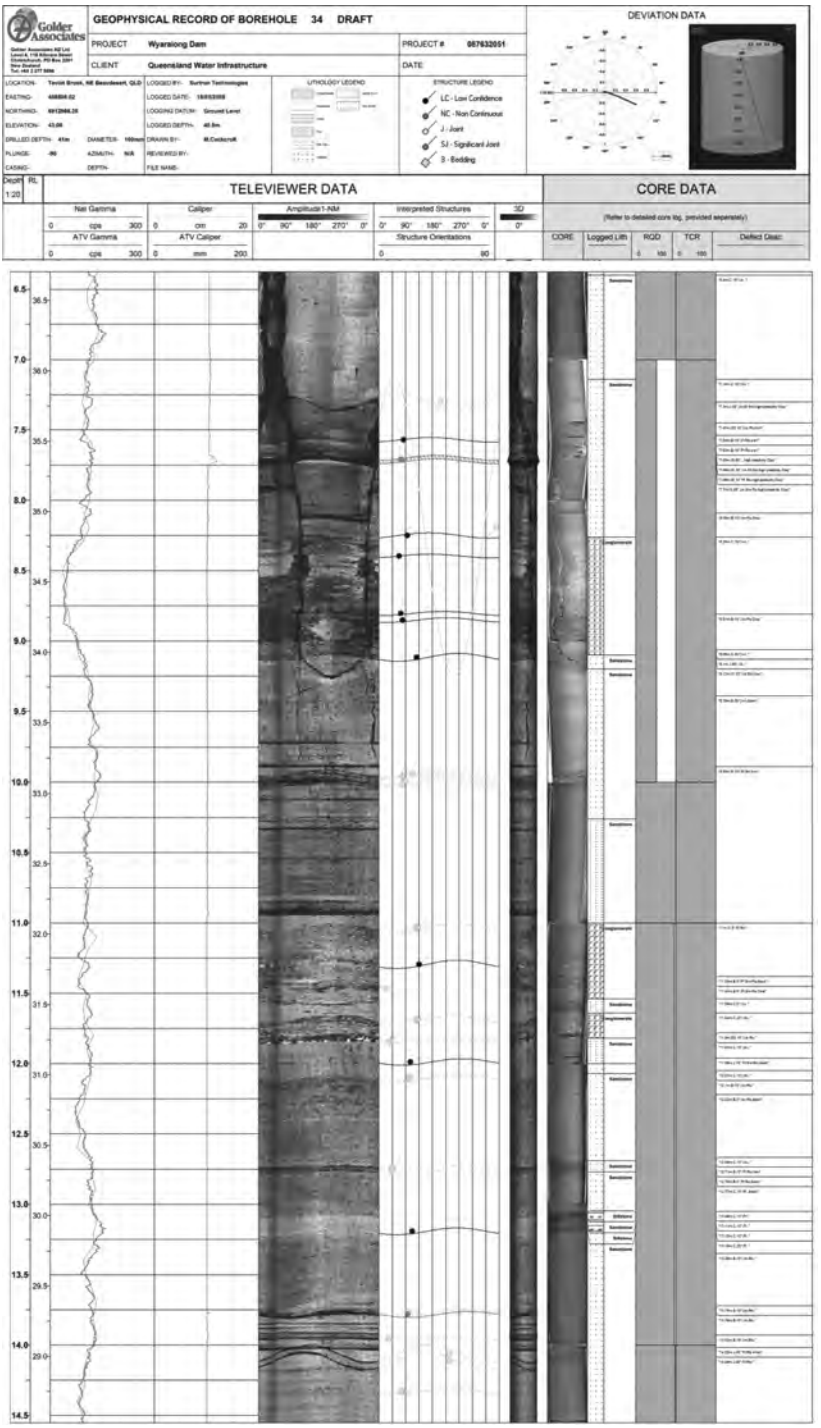


Figure 5.32 Example of an acoustic televiewer log (Courtesy of SEQ Water and Golder Associates).

5.12 GROUNDWATER

In all aspects of the investigation for dams the groundwater situation should be continually monitored. This involves the location and monitoring of seepage points during surface mapping, recording of groundwater inflows into pits and trenches, and the monitoring of groundwater levels during the drilling investigation. If at all practicable, some of the monitoring points should be organized to cover the first-fill and later in-service periods. As full a history as possible of the groundwater monitoring can be a great help in any review of the dam. Piezometers (with sealed tips and not just open slotted PVC 'wells') should be installed in drill-holes (see Chapter 20) and regularly measured throughout the project. The response of groundwater levels to rainfall can provide a useful indication of mass permeability.

Measurement of seepage and flow rates, recording of rainfall, evaporation, and water levels installed in exploration drill holes gives a broad indication of the proportion of rainfall which infiltrates, the response of the groundwater to storms, and areas where high groundwater flow rates may be expected. This programme provides a relatively low cost general picture of the regional hydrogeology which can be refined by an investigation of specific permeability values at individual sites.

Claims related to unforeseen groundwater conditions form a significant proportion of contractual disputes. Many of these claims originate from a failure to record adequate groundwater information during site investigations and during the period between the completion of investigations and the start of construction.

Figure 5.33 shows how a perched and main water table can be inferred from the water levels measured in a borehole during drilling using the DVD (depth of hole versus depth of water) plot.

5.13 *IN SITU* PERMEABILITY TESTS ON SOIL

The permeability of soil in a dam foundation may be of importance if the cutoff is founded in the soil, as in dams on alluvial, colluvial and glacial deposits and on some deeply weathered residual or lateritised soils.

In general the structure of the soil controls water flow, for example:

- sandy layers in alluvial soils,
- root holes and fissures in residual and alluvial soils,
- worm burrows in alluvial and residual soils,
- leached zones, infilled root holes and relict joints in lateritised soil and extremely weathered rocks.

Hence it is seldom possible to obtain a realistic estimate of soil mass permeability from laboratory testing. Some of the difficulties associated with laboratory testing are discussed in Chapter 6.

Because the soil structure tends to be blocked or smeared by the drilling action, pump-in type permeability tests in boreholes can also give quite misleading results, with measured permeability values one or two orders of magnitude lower than the actual soil mass permeability.

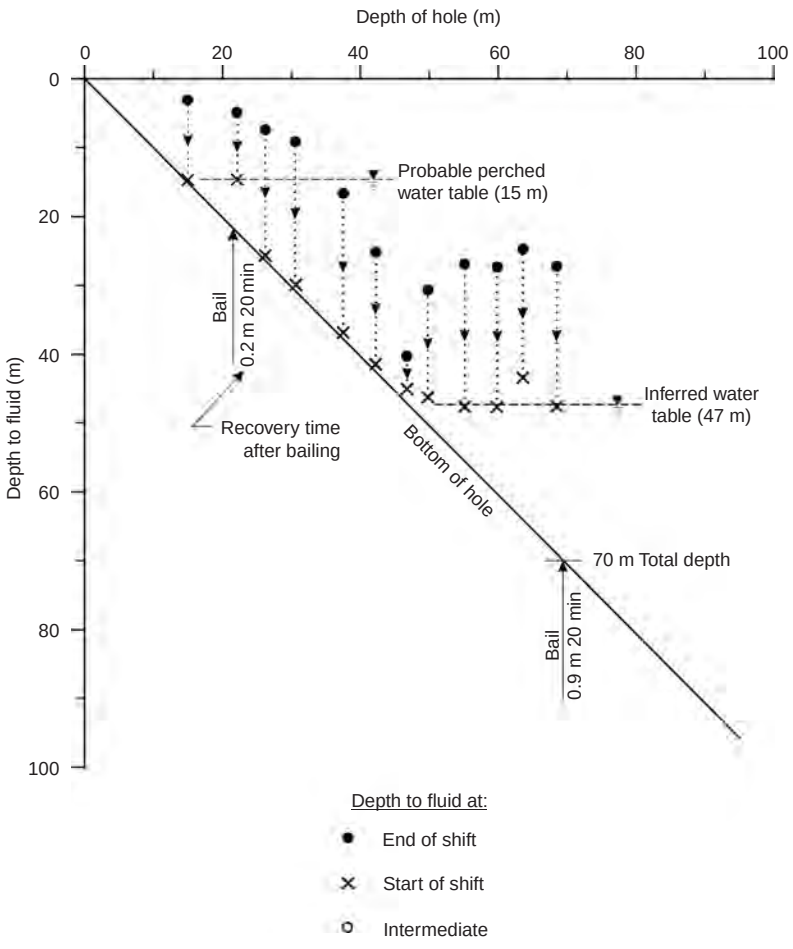


Figure 5.33 Method for estimating perched and main water table levels from monitoring during drilling.

Where possible, pump out tests should be conducted with a pump well and observation wells. Figure 5.34 shows a typical arrangement.

In a pump out test the inflow of water removes the soil blocking or smearing the hole side and realistic estimates of permeability can be obtained.

The design of the pump well, observation wells and the test program analysis need to recognise the effect of different vertical and horizontal permeabilities in the soil. The results are often difficult to assess and it is recommended that a groundwater hydrologist be employed in the planning, execution and analysis of such tests.

In the event that the groundwater level is below the depth of interest, it is necessary to carry out pump-in tests. Even when considerable care is taken to clean the hole, it should be understood that the results of such tests give lower bound estimates and that the actual permeability may be up to one or two orders of magnitude greater than indicated by the test result.

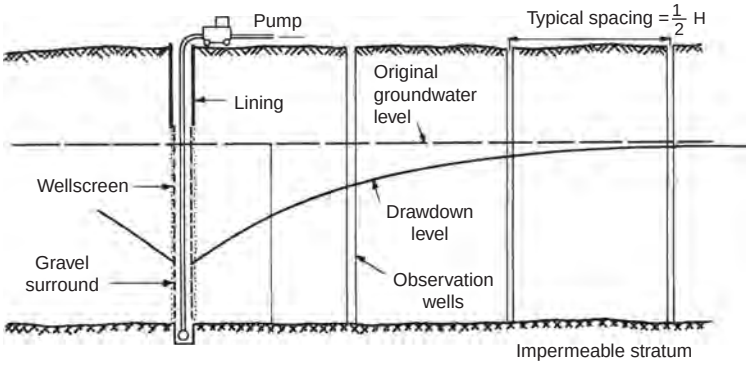


Figure 5.34 Typical arrangement for a well pumping test with observation wells.

5.14 *IN SITU* PERMEABILITY TESTS IN ROCK

5.14.1 Lugeon value and equivalent rock mass permeability

The permeability of the rock mass can be determined by either constant head tests or falling head tests. A section of a drill hole is isolated using a sealing packer and water is added to maintain a constant head, or the rate of fall in water level is measured after a slug of water is added to the hole. Both methods suffer from the potential effects of smear and clogging of defects, but by careful flushing of the hole before testing, reasonable values can be obtained. In falling head tests the additional pressure which can be added to the test section is limited by the level of the test section and the practicability of extending the pipe imposing the head above ground level.

The most common and effective method of measuring rock mass permeability is the water pressure test (also known as the Lugeon or 'packer' test). The test consists of isolating a section of drill hole and pumping water under pressure into that section until the flow rate for any given pressure is constant (i.e. it is a constant head test). The use of successive rising and falling test pressures establishes the relationship between the volume of water accepted into the section and the pressure, to provide an estimation of permeability, and indicate water flow mechanisms.

As rock substance is generally almost impermeable, the permeability determined in this test represents an indication of the number, continuity and opening of the rock defects which intersect the wall of the borehole in the test section.

Results are expressed in Lugeon (uL) units. A Lugeon is defined as the water loss of 1 litre/minute per metre length of test section at an effective pressure of 1 MPa.

Indicative rock permeabilities are:

Lugeon	*Range*	*Condition*
<1	Low	Joints tight
1–5	Low/Mod.	Small joint openings
5–50	Mod./High	Some open joints
>50	High	Many open joints.

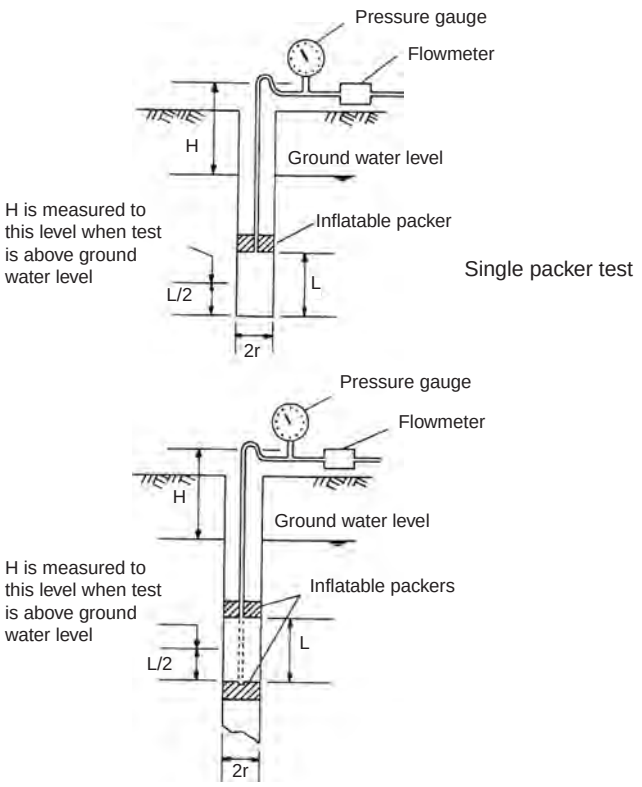


Figure 5.35 Packer permeability testing; single packer and double packer methods.

There is no unique relationship between Lugeon value and equivalent rock mass permeability (ke). Moye (1967) recommended use of the equation:

$$ke = \frac{QC}{LH} \tag{5.1}$$

where ke = the equivalent coefficient of permeability (m/sec)
Q = the flow rate (m^3/sec)
L = the length of the test section (m)
H = Net head above the static water table at the centre of the test section (m)

$$C = \frac{1 + \ln(L/2r)}{2\pi} \tag{5.2}$$

r = the radius of the hole (m).

This is based on the assumption of radial laminar flow in a homogeneous isotropic rock mass, a condition seldom, if ever, achieved.

Hoek and Bray (1981) suggest the use of the equation:

$$ke = \frac{Q \ln(2mL/D)}{2\pi LH} \tag{5.3}$$

where $m = (ke/kp)^{1/2}$
kp = equivalent permeability parallel to the hole
ke = equivalent permeability normal to the hole
D = diameter of hole.

They suggest that for most applications ke/kpis about 10^6. This implies no fractures parallel to the hole and in most rocks would not be a reasonable approximation.

If it is assumed that ke/kp = 10, and water pressure testing in 5 m lengths of NMLC hole (75 mm diameter), then 1 Lugeon is equivalent to $ke = 1.6 \times 10^{-7}$ m/sec.

For ke/kp = 1, i.e. homogeneous, isotropic conditions, m = 1 and 1 Lugeon is equivalent to $ke = 1.3 \times 10^{-7}$ m/sec. The Moye (1967) formula gives similar results to this homogeneous isotropic case.

5.14.2 Test methods

There are two common methods of water pressure testing in a drill hole (Figure 5.35). The 'down-stage' (or 'single-packer') method is recommended and involves isolating and testing successively the bottom sections of the drill hole. This method enables progressive assessment of permeability and allows later stabilization of the wall of the hole by casing or grouting if caving occurs. A disadvantage of down-stage packer testing is that it disrupts drilling progress, but this is far outweighed by its advantages over the alternative method discussed below.

The alternative method is to complete the drilling of the hole and water test in sections by sealing the hole above and below the test area (the 'double-packer' method). This method has the advantage of convenience in that all water testing is carried out at one time but results can be affected by:

- Damage to the sides of the hole by drill rods and casing,
- Possible leakage from the test section past the lower packer, which cannot be detected and
- Sections of the hole which have been stabilised by cement or by casing to enable deeper drilling cannot be tested. Commonly these sections will be highly permeable.

The purpose of the test is to estimate the potential of water to pass through rock defects. The use of drilling mud to stabilise the hole can block these defects and make the results of water pressure testing meaningless. For best results the drilling fluid used in holes where permeability testing is required should be water. If necessary a small amount of soluble oil appears to improve drilling efficiency without affecting permeability.

There are several proprietary chemical drilling mud such as 'Revert' which are reputed to break down and dissolve when treated with 'Fastbreak'. Testing experience indicates that this procedure is only partly effective and that holes drilled with Revert give lower indicated permeabilities than holes in the same situation drilled with water.

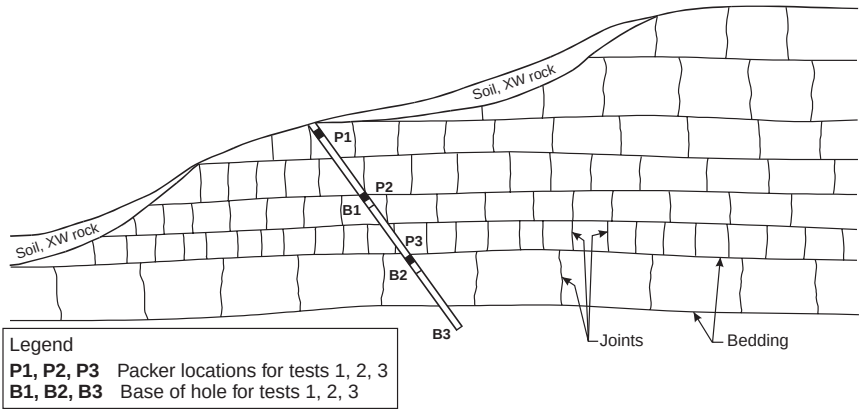


Figure 5.36 Borehole inclined to intersect joints in the rock.

5.14.3 Selection of test section

To maximize the information on rock mass permeability the investigation drill hole should be oriented to intersect as many joints as practicable (see Figure 5.36).

Every effort should be made to test the total length of hole in rock. It is preferable to overlap sections and thus have two tests over a short length than to miss some length of hole.

The upper limit which can be tested is the highest level at which a packer can be satisfactorily sealed, often in distinctly weathered rock. Location of the packer within casing above the rock does not seal the hole as water may leak past the casing.

The length of test section depends on the nature of the rock defects and the type of structure under investigation. Examination of the drill core usually indicates the presence of typical fracture spacing which represents probable background permeability and anomalous structural features which may be associated with zones of higher permeability.

Test sections should be selected to provide an indication of the relative width and permeability of these zones. In many cases water flow may be concentrated through a few fractures. The water loss is averaged over the length of the test section and, if the section is too long, the presence of a high permeability zone may not be recognised.

Usual test section lengths range from 3 m to 6 m but the length may be increased in essentially un-fractured rock. When a particularly high water loss is recorded it is good practice to repeat the test over a shorter section of the hole to further define the zone of high permeability.

5.14.4 Test equipment

The equipment required for water pressure testing includes:

– a packer to seal the test section,
– a water line from the packer to the supply pump,

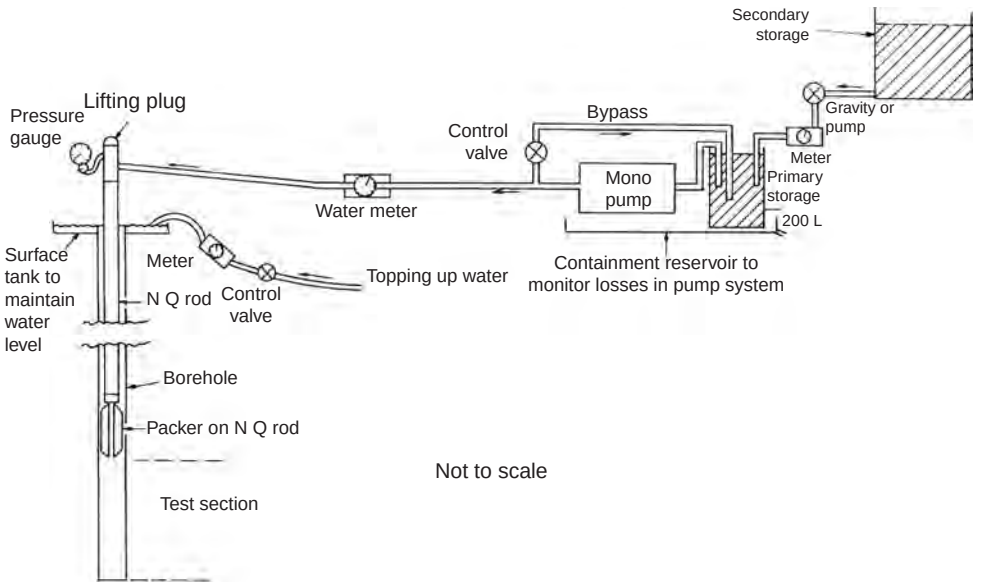


Figure 5.37 Layout of packer test equipment.

- a pump to supply water under pressure,
- a bypass to control the pressure,
- a surface tank at the borehole to maintain the water level outside the water line,
- water storages,
- a pressure gauge,
- a water meter.

A layout used for water pressure testing is shown on Figure 5.37.

5.14.4.1 *Packers*

There are several types of packer used:

- A *hydraulic packer* (Figure 5.38) consists of a double tube with rubber sleeve. When the packer is in the 'down' position water can be pumped into the sleeve to inflate the packer and seal the hole. In the 'up' position water flows into the test section.

  Hydraulic packers have proved reliable under most conditions. It is necessary to ensure that the hole outside the water line is kept full, as outflow from this area indicates possible leakage past the packer, and also it is necessary to equalise the pressures and allow effective inflation and deflation of the sleeve.
- In a *pneumatic packer* the sleeve is inflated by air from a compressed air bottle using a separate air line to the surface. This method is effective in shallow holes, but with deeper holes fracturing of the air line has caused problems. Water levels do not affect inflation or deflation.

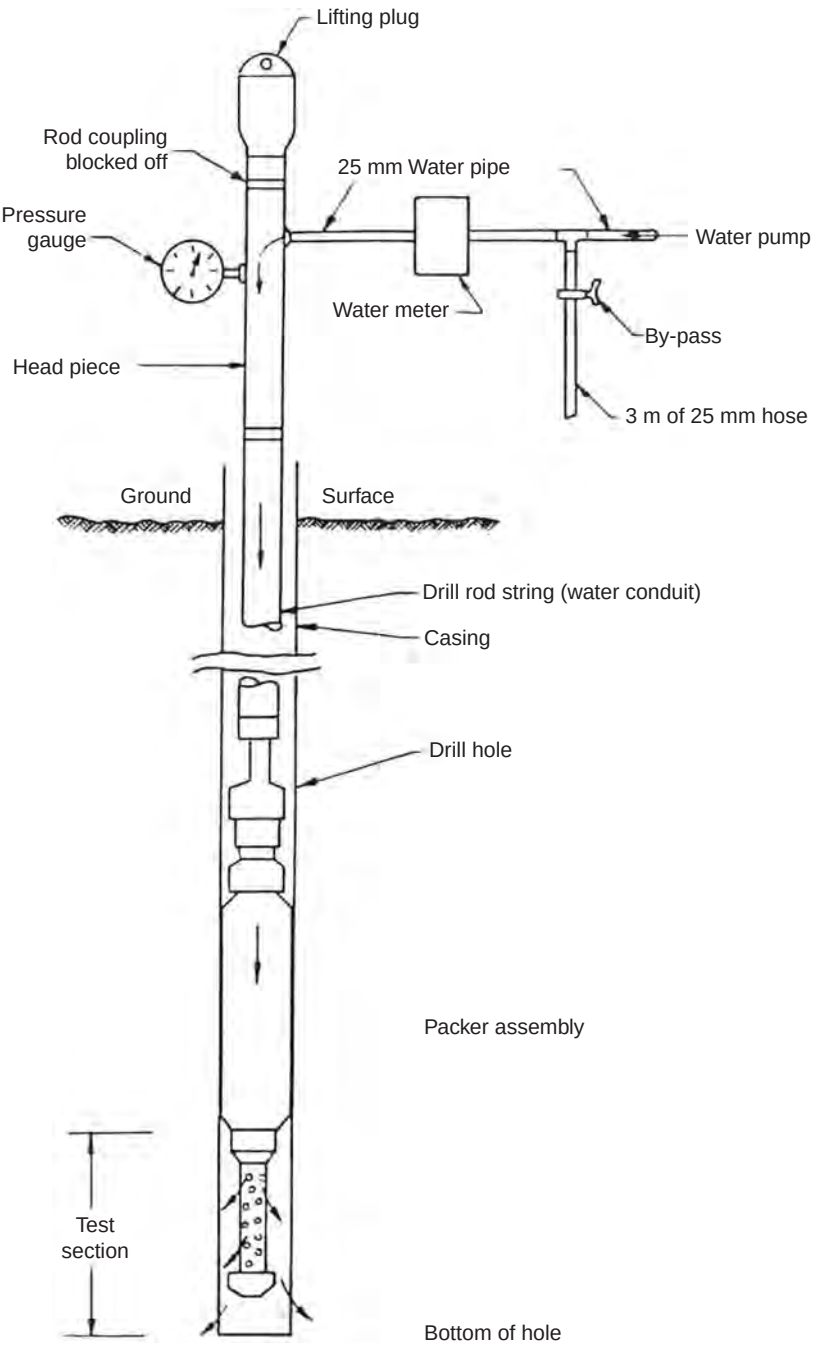


Figure 5.38 Hydraulic packer equipment (Coffey Geosciences).

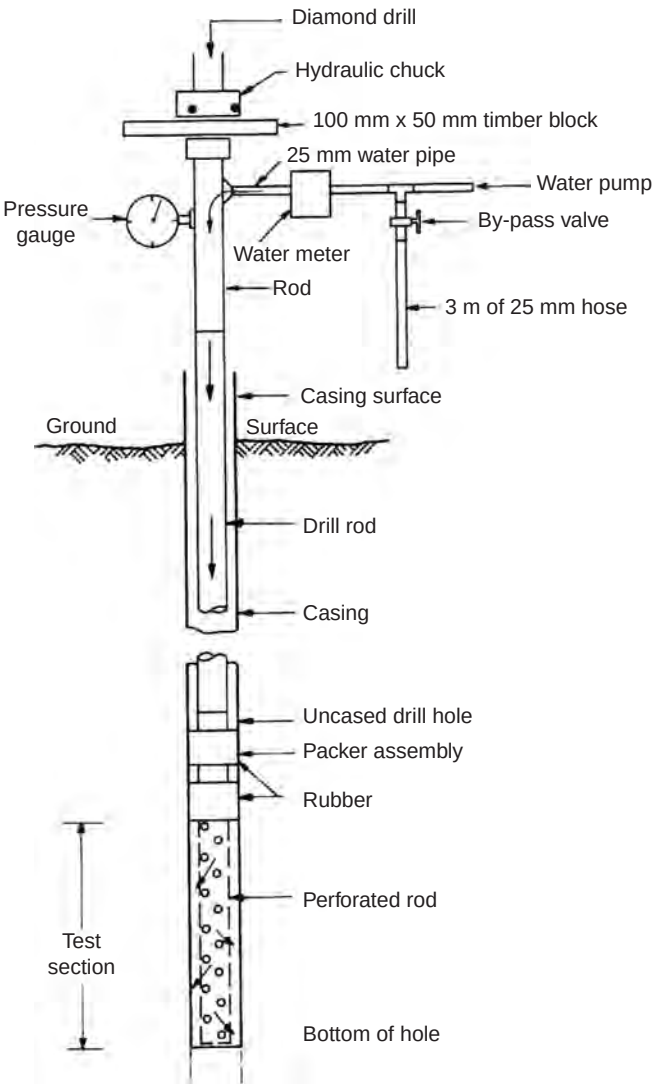


Figure 5.39 Mechanical packer equipment (Coffey Partners International).

– *Wireline packers* have been developed for testing holes drilled with wireline equipment. These enable water pressure testing without the withdrawal of the drill rods. The packer incorporates two sealing sleeves, the upper seal within the drill rods and the lower seal in the hole below the drill bit.
– The *mechanical packer* seals by the expansion of two or more rubber rings when compressed (Figure 5.39). The test section length is controlled by the insertion of a selected length of perforated rod below the packer. Sealing is achieved by downward pressure on the drill string by the drill chuck. This pressure must be maintained throughout the test.

The capability of the rubber rings to expand is limited. With a sealing length of about 200 mm the formation of an effective seal in closely fractured rock is difficult. Alteration of the test section length involves removal of the whole drill string and the addition or removal of rods below the packer. This operation is time consuming in deep holes. Mechanical packers do however have the advantage of fewer operational problems.

A double mechanical packer requires complete withdrawal and addition or removal of rods below the bottom packer between each test.

It is considered that mechanical packers should only be used where other packers are not available and results are not critical. Double packers should be avoided if at all possible.

5.14.4.2 *Water supply system*

The line connecting the packer to the surface should be watertight with minimum restriction to water flow. Flush coupled rods or casing are commonly used. The test water should be clean and supplied by a pump with bypass to enable control of pressure. Dirty water will probably result in clogging of fractures and lower Lugeon values than are correct for the rock.

A centrifugal pump or reciprocating pump with surge chamber is necessary to ensure constant pressure. Pressure and water flow is measured by meters which should be recently calibrated. Before testing the water supply rods and packer should be calibrated for friction losses at different flow rates.

5.14.4.3 *Selection of test pressures*

The object of the test is to:

- measure the natural permeability,
- indicate the probable water flows under the expected hydraulic loading by the proposed structure.

The pressures applied during the test should not be sufficient to produce hydraulic fracturing (jacking) of the rock around the test section. In weak rocks near the ground surface this fracturing does take place at relatively low pressures and is usually indicated by an unexpected increase in water loss following a raising of test pressures. It is recommended that, to avoid potential 'jacking', maximum effective test pressures be limited below overburden pressure (approximately 22 kPa/m). In low strength weathered rocks, lower values will probably be necessary.

5.14.5 Test procedure

The test involves:

- Measurement of ground water level.
- Washing out of drill hole. Circulation of drilling water should be continued for at least 15 minutes after the water appears clear.
- Installation of packer at the selected level.

Table 5.6 Sample results of water pressure test Hole No.: JB24; Test Section: 20.05 m to 26.50 m; Section Length: 6.45 m.

Duration of test mins	*Gauge pressure kPa*	*Water loss litres*	*Litres per min*	*Litres per min/m*
5	20	20.0	4.0	0.62
5	20	19.0	3.8	0.59
5	60	28.5	5.7	0.89
5	60	28.5	5.3	0.82
5	110	36.0	7.2	1.12
5	110	35.0	7.0	1.08
5	170	48.5	9.7	1.50
5	170	46.5	9.3	1.44
5	110	34.0	6.8	1.05
5	110	32.0	6.4	0.99
5	60	25.0	5.0	0.78
5	60	25.5	5.1	0.79
5	20	15.5	3.1	0.48
5	20	16.0	3.2	0.50

Interpreted result = 6 lugeons.

– Connection of the water supply system.
– Application of the test pressures and measurement of water loss.
– Removal of equipment.

The testing is carried out in several stages with different pressures. Commonly at least three test pressures are used (five are desirable). Pressures are applied in an increasing and then decreasing sequence. For example with three pressures – a, b, c – the water loss is measured at stages with pressure successively at a, b, c, b, a. Each stage should be continued until a constant rate of water loss (within 10%) for a 5 minute period is recorded.

5.14.5.1 *Presentation and interpretation of results*

The results are best plotted with the effective test pressure at the centre of the test section against the flow rate. The effective test pressure is the gauge pressure corrected for the elevation difference between gauge and water table and for friction losses in the system. (i.e. H_T in Figure 5.35). Where the water level is not known, or suspected to be locally elevated due to the effects of the testing, the results are plotted using gauge pressure only. Typical test results (Table 5.6) plotted in this way are shown in Figure 5.40. Note that the data from the increasing pressures do not plot identically to those on the decreasing pressures. The plot is judged to be parallel to the 6 Lugeon line. A range of different results is possible, indicative of the following possible mechanisms:

– Laminar flow and no change in permeability during the test.
– The occurrence of turbulent rather than laminar flow.
– Scour of joint infill, weathered or crushed rock.

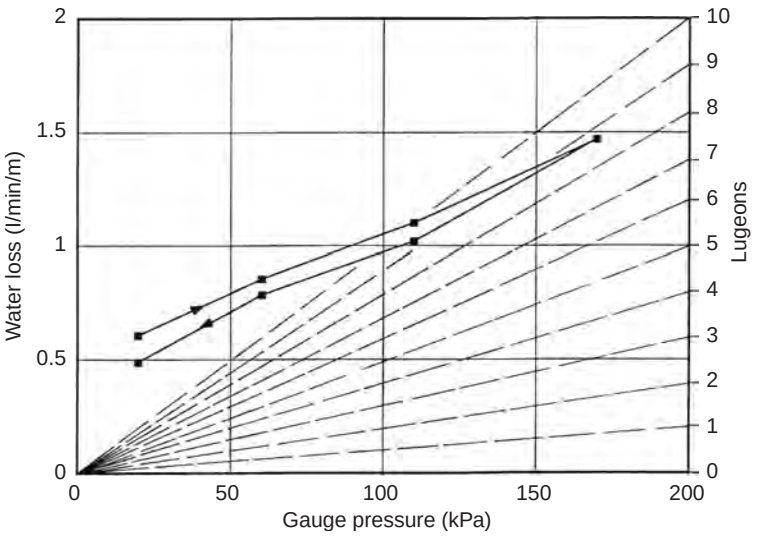


Figure 5.40 Typical results of packer test.

- Leakage past the packer.
- Sealing of joints by fines eroded by the water.
- Hydraulic fracture of the ground.
- Inaccuracies in measurement.

Figure 5.41 shows examples of plots indicating these types of behaviour.

5.15 USE OF SURFACE SURVEY AND BOREHOLE INCLINOMETERS

5.15.1 Surface survey

In the investigation of dam sites and the assessment of existing dams the precise measurement of location and elevation of the ground surface provides information on:

- Existing mass movement – e.g. landsliding
- Foundation settlement
- Deformation or displacement of structures.

A range of surface survey methods is included in Table 5.7.

The development of computer assisted survey instruments has enabled the determination of the position of many surface points to be carried out rapidly and with great accuracy. A vital component of any dam investigation should be the establishment and regular measurement of a network of surface stations. Survey marks should be founded below the level of reactive soil and away from traffic. The points should be clearly marked and numbered.

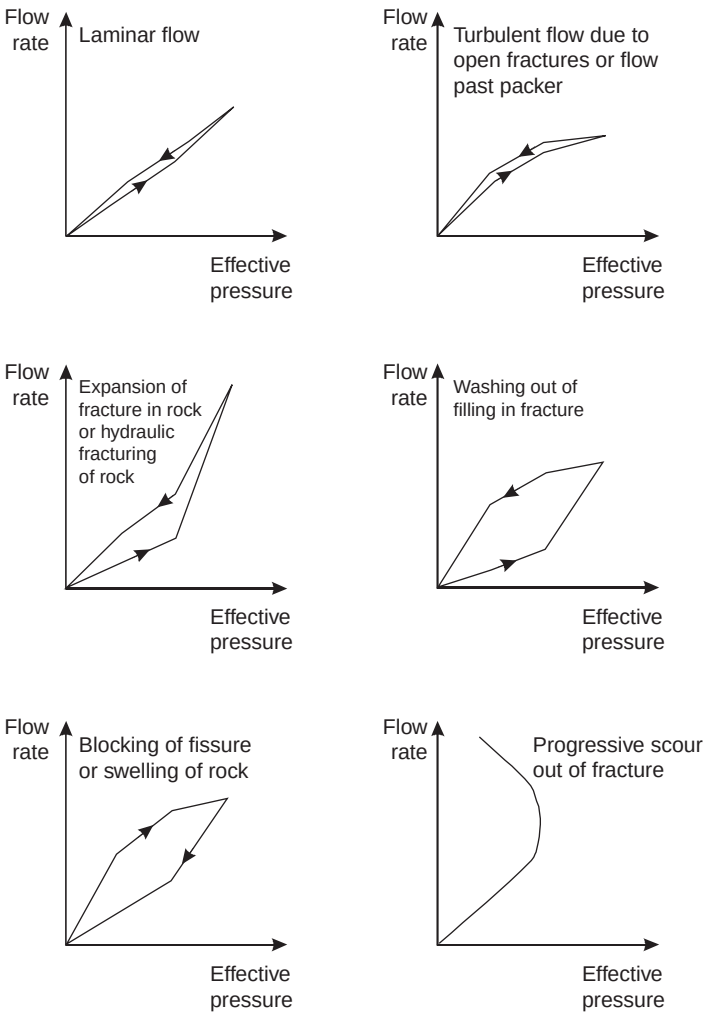


Figure 5.41 Typical effective pressure versus flow curves for packer tests.

The results of repeat surveys of these marks should be recorded as amount of differential movement and plotted as vectors of vertical and horizontal displacement. These plots enable a quick assessment of the amount and consistency of the movements that have been recorded.

The authors experience is that often the most valuable data in detecting marginal instability, and internal erosion and piping in existing embankments and foundations is long term deformation monitoring. Any review of dam safety should include a review of deformation data for the embankment and associated structures such as retaining walls supporting the embankment at the spillway.

Table 5.7 Instrumentation and methods for the measurement of surface and subsurface displacement (adapted from Dunnicliff, 1995 and Fell *et al.*, 2000).

Parameter	*Instruments/Methods*
Surface deformation	Surveying methods, including GPS
	Crack gauges/surface extensometers
	Tilt meters
	Multi-point liquid level gauges
	Photogrammetry
	Satellite images
	Remote Video
Subsurface deformation	Inclinometers
	Simple borehole deformation measurements
	Fixed borehole extensometers
	Slope extensometers
	Shear pin indicators
	In-place inclinometers
	Multiple deflectometers
	Acoustic emission monitoring
	Time domain reflectometry (coaxial cables)
	Pendulum

5.15.2 Borehole inclinometers

Subsurface movement can be measured using several different instruments as listed in Table 5.7 and discussed in Chapter 20. These instruments are also used in the study of landslide movements and are described by Dunnicliff (1995).

The most common method uses a borehole inclinometer, the principles of which are shown in Figure 20.27. Successive readings on a borehole inclinometer provide information on the amount and direction of deflection and the location of the point of rupture (Figure 5.42) A borehole extensometer uses magnets which can be installed outside the PVC casing in soil or rock (Figure 20.23). Sliding couplings in the casing allow relative movement which can be measured using a probe lowered down the borehole.

The installation of electrical shear strips within a slope or embankment when monitored by a readout unit can indicate the location of an area of slope movement but not the amount of movement.

5.16 COMMON ERRORS AND DEFICIENCIES IN GEOTECHNICAL INVESTIGATION

Many problems during design and construction of dam structures are caused by poor quality or inadequate geotechnical investigation. This is often due to the investigation not following good engineering practice. Many of the problems are known to competent practitioners. In some situations lack of available finance can force the adoption

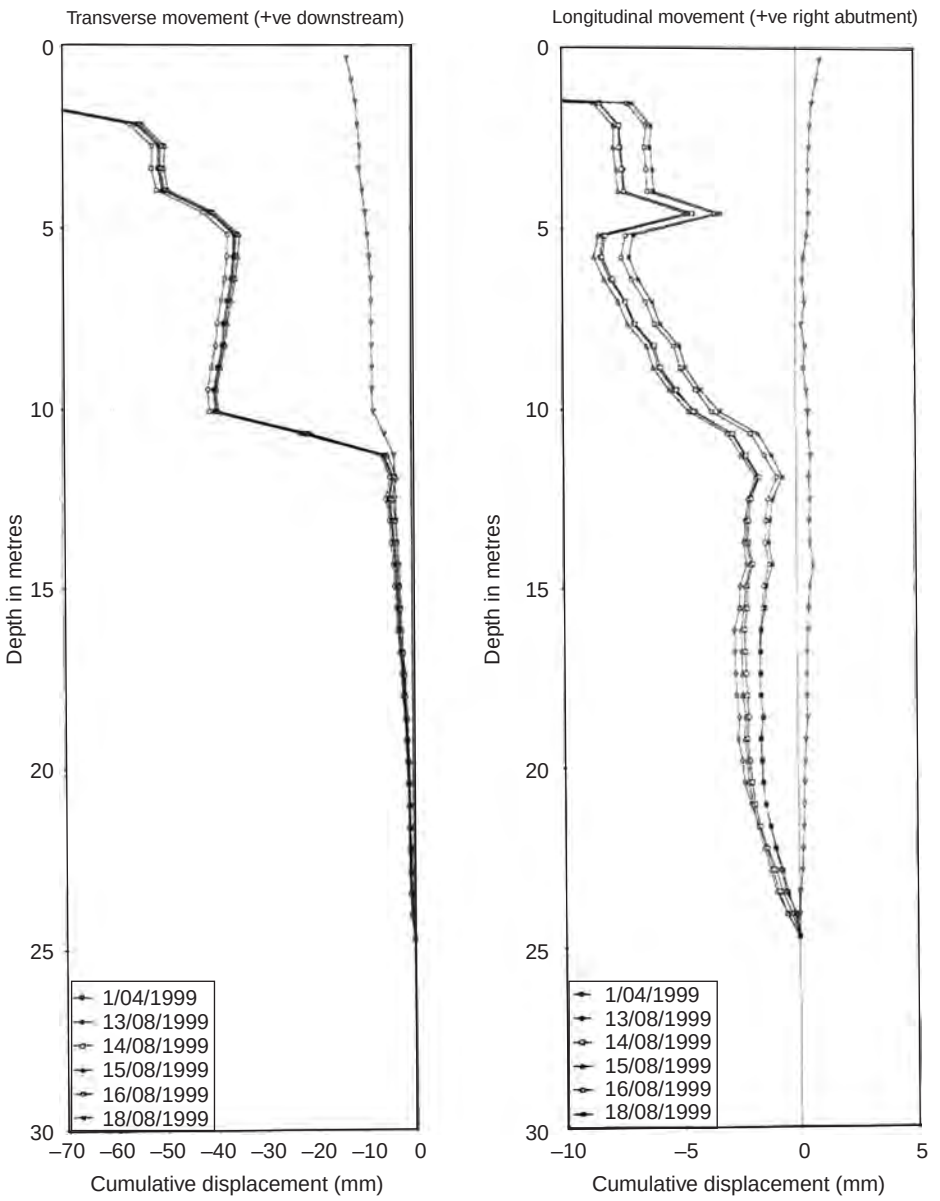


Figure 5.42 Plot of borehole inclinometer readings.

of poor investigation practice. In many cases these inadequacies only become apparent during construction and lead to costly redesign and/or contractual disputes. Tables 5.8 to 5.12 list some specific problems that occur in geotechnical investigations, the consequences, and measures which can be taken to avoid the problems.

Table 5.8 Deficiencies in drilling and sampling.

Problem	*Consequences*	*Remedy*
Only drilling vertical holes where jointing is near vertical	Joint spacing and rock mass permeability incorrectly assessed	Angle holes to intersect joints, bedding and other features
Poor identification of orientation of joints in boreholes	Lack of knowledge of actual joint orientation affecting slope stability	Careful orientation in boreholes and mapping in trenches
Water table not measured, not measured often enough or data not recorded	Dewatering problems with resultant contractual claims; lack of information for design, e.g. pore pressures for slope stability	Routine measurement and or recording. Install casing and or piezometers
Drill water inflows or outflows not recorded	Poorer understanding of reasons for high/low water tables	Employ good drillers and supervise full time
Poor identification of layering in soil deposits particularly when using augers below water table, or wash boring without sampling, e.g. see Figure 5.14 which compares log from augered borehole and static cone penetration test	Soil strata are mixed together, hole may collapse causing greater mixing. Leads to under/ over-estimation of horizontal/ vertical permeability, potential overestimation of strength by mixing clay layers	Use rotary drilling with mud and/or casing, sample systematically with thin wall tubes and SPT. Use static cone penetrometers
'Blowing' in boreholes in silty sand, sand and soft clay i.e. flow of material towards borehole	Low SPT values in silt and sand and disturbed samples (in clay) leads to over-estimation of settlement, underestimation of strength	Use drilling mud and excess head of mud in the borehole to prevent blowing
Only drilling holes, not test pits, in 'structured' clays, e.g. fissured or lateritised soils, soils with root holes	Failure to recognize the structure usually leads to overestimation of strength and underestimation of permeability (often by orders of magnitude)	Dig backhoe and excavator pits and have experienced personnel log them
Drilling in gravel and gravelly sands with percussion drill	Gravel is broken up to finer particles, mixed with sand to give the impression of uniform sandy gravel. Fines may be lost in the drilling process. Horizontal permeability is under-estimated, possibly by orders of magnitude (see Figure 6.62)	Recognise the problem. No real drilling solution. Test permeability with pump out tests
Ground surface level and location of boreholes not surveyed	Errors in plotting sections, plans misinterpretation of conditions	Survey all investigation location

Table 5.9 Deficiencies in *in situ* testing.

Problem	*Consequences*	*Remedy*
Testing only at predetermined depths, e.g. 1.5 m, 3 m, 4.5 m etc., e.g. SPT, undisturbed tube sampling	Poor identification of strata, poor selection of strata to test and sample	Supervise full time and test at strata changes as well as at pre-determined depths
SPT (and CPT) in gravelly soils affected by coarse particles	Overestimation of SPT 'N' value, with resultant overestimation of relative density, underestimation of compressibility	No real remedy. Just recognise the problem or seek other ways of estimating the parameters
Not washing borehole carefully before water pressure testing	Joints remain clogged with drill cuttings, Lugeon value underestimated	Take care in washing hole. Use clean water for testing
Lack of *in situ* permeability tests in soils	Contractual claims because 'conditions are worse than contractor assumed'; gross errors in estimation of permeability	Do appropriate *in situ* tests despite the costs involved
Estimation of permeability from particle size distribution	Underestimation of permeability because of mixing of finer layers. See Figure 6.62	Sample each stratum, and only use 'Hazen' type formulae in uniform, clean fine-medium sand for which it was derived. Do laboratory permeability tests.
Use of pump-in permeability tests in soil, particularly structured clay and in augered boreholes	Gross underestimation of permeability due to smearing and clogging of fissures, root holes, sandy layers, (by factor of 10 to 103).	Use pump-out tests where soils are below water table. Above water table use pits, with the sides carefully cleaned to remove smearing. Adopt 'realistic' values for design regardless of results
Use of seismic refraction survey to estimate rippability	Incorrect prediction of rippability, contractual claims	Do seismic refraction correctly (see Whiteley, 1988). Couple with geological factors (see MacGregor *et al*., 1994 and Stapledon, 1988b), and recognise estimates are approximate in contractual arrangements
Installation of 'wells' instead of properly constructed piezometers in boreholes	Measures phreatic surface, not pore pressure. May over/ underestimate pore pressures (see Figure 20.15 and Fell, 1987)	Install piezometers, properly sealed in borehole

Table 5.10 Deficiencies in logging of boreholes, test trenches and pits.

Problem	*Consequences*	*Remedy*
Logging rock in soil description terms, e.g. 'black silty clay' in holes drilled by auger, with the rock ground to soil consistency by the drilling bit	Contractual claims relating to difficulty of excavation; incorrect design assumptions	Use correct drilling techniques and/or log correctly and/or log in soil and rock terms
Logging joints, partings and drill breaks all together as 'fractures'	Incorrect assessment of the joint spacing leading to incorrect assessment of ease of rippability and size of ripped rock and contractual claims; incorrect assessment of slope stability and grouting conditions	Log joints, bedding plane partings and drill breaks separately and present data clearly
Failure to log condition of joints, e.g. clay coating, iron stained, and continuity of points. Poor definition of weathering classification	Incorrect assessment of joint strengths in slope stability, and of likely flow rates of water, grouting conditions, rippability. Confusion on acceptable rock conditions during construction with resultant contractual claims	Use experienced personnel to log core (or at least check logging). Define weathering classification and/or define acceptable conditions accurately
Incorrect description of cemented soils, either failing to describe the cementing, e.g. in calcareous sands OR describing cemented soil as rock	Incorrect assessment of conditions for design and contractual claims for excavation or tunnel support conditions	Inspect exposures in large cuttings, dig pits, relate drilling results to the local geology and log accordingly
Failure to log soil structure, e.g. fissures, root holes, minor inter-bedding	Incorrect assessment of shear strength of fissured soils, underestimation of permeability	Log carefully and systematically in pits
Classifying soils in the dry state	Underestimation of clay content and plasticity leading to incorrect design specification and contractual claims	Moisten soil before classifying
Incomplete description of soil, e.g. omission of moisture conditions, consistency, colour	Incorrect assessment of conditions for design, specification and contractual claims based on unforeseen 'wet soil' etc.	Log carefully and systematically
Inadequate description of 'organic matter' and 'fill'	Contractual claims and incorrect design	Describe in detail in the logs

Table 5.11 Deficiencies in data presentation and interpretation.

Problem	*Consequences*	*Remedy*
Too much detail on irrelevant features, e.g. lengthy description of trench logs, or mineralogy of rock types in borelogs. Confusion of 'facts' and interpretation	Important features lost in the mass of irrelevant data. Misinterpretation of geological conditions, potential for contractual claims	Proper planning and briefing of personnel by experienced geotechnical practitioners. Clear distinction and definition of terms
Straight line interpolation between boreholes without regard for lack of data or geological conditions	Misinterpretation of geological conditions, overconfidence in interpolation, potential for contractual claim	Draw interpretive sections with due allowance for geology, e.g. core stones in granite, buried land surfaces
Use of exaggerated scales in preparing sections	Misinterpretation of geological conditions, overconfidence in interpolation, potential for contractual claim	Use natural scale or provide both natural and exaggerated scale
Consideration of data on a hole to hole basis	Misinterpretation of geological conditions, incorrect assessment of design parameters and the range of values	Determine a proper geotechnical model of the site based on all data
Failure to recognise that it is the exception which sometimes causes the problems, e.g. a thick bed of rock or a thin bed of high strength rock causes problems with rippability and size of ripped material	Contractual claims	Recognise the importance and include in reports and interpretive sections
Incorrect projection of information from boreholes, in particular altering levels	Incorrect interrelation of conditions	Project along contour and/or with consideration of geological controls

Table 5.12 Deficiencies in earthfill borrow area investigations.

Problem	*Consequences*	*Remedy*
Use of boreholes to investigate earthfill and sand/gravel deposits	Failure to recognise variability of soils, water contents incorrectly determined, contractual claims	Use backhoe or excavator pits where possible ($<$6 m)
Lack of water content profiles, or profiles taken in periods not typical of contract	Lack of knowledge of moisture condition requirements, contractual claims	Take profiles in period representative of construction
Lack of information on 'bulking' factors in earthworks	Deficiency of material when 'shrinkage' occurs leading to contractual claims	Do density in place tests and laboratory compactions in borrow area investigations
Variability of alluvial and non alluvial soil deposits	Difficulty in selecting materials during construction, contractual claims	Dig plenty of test pits

Chapter 6

Shear strength, compressibility and permeability of embankment materials and soil foundations

6.1 SHEAR STRENGTH OF SOILS

6.1.1 Drained strength – definitions

When a soil is sheared slowly in a drained condition (so that there is sufficient time for dissipation of pore pressures induced by shearing), the stress displacement curves will take the general form shown in Figure 6.1.

The behaviour is dependent on whether the soil has a high or low clay fraction (finer than 0.002 mm) content and on whether it is normally consolidated (NC) or over consolidated (OC), but has the following common features:

- A peak strength is obtained at a small displacement.
- A reduction of strength to the critical state or fully softened strength then occurs with further displacement. For overconsolidated soils this is due to increase in water content with dilation of the soil as it is sheared. The fully softened strength corresponds to the critical state (Skempton, 1985), i.e. when continuing displacement occurs without further change in volume or water content. Note that normally and overconsolidated samples of the same soil will tend to achieve the same critical state or fully softened condition.
- With continuing displacement of soils with a high clay fraction content, particle reorientation occurs, resulting in a further reduction in shear strength. The minimum value of shear strength achieved at large displacements is the 'residual' strength.
- The residual and fully softened strengths are significantly different for high clay fraction content soils, but not for soils with low clay fraction content (see Figure 6.1). Sands behave similarly to the low clay fraction soils.
- Strength envelopes for these cases are defined by:
  - peak strength $c'\phi'$
  - softened strength $c'_s\phi'_s$
  - residual strength $c'_R\phi'_R$

Note that often $c'_s \approx c'_R \approx 0$, and $\phi'_s \approx \phi'$. Figure 6.2 shows the relationship between these parameters.

Skempton (1985) adopts the use of the term 'field residual' strength (ϕ'_R) as the strength of fully developed shear or slide surfaces in nature. As discussed in

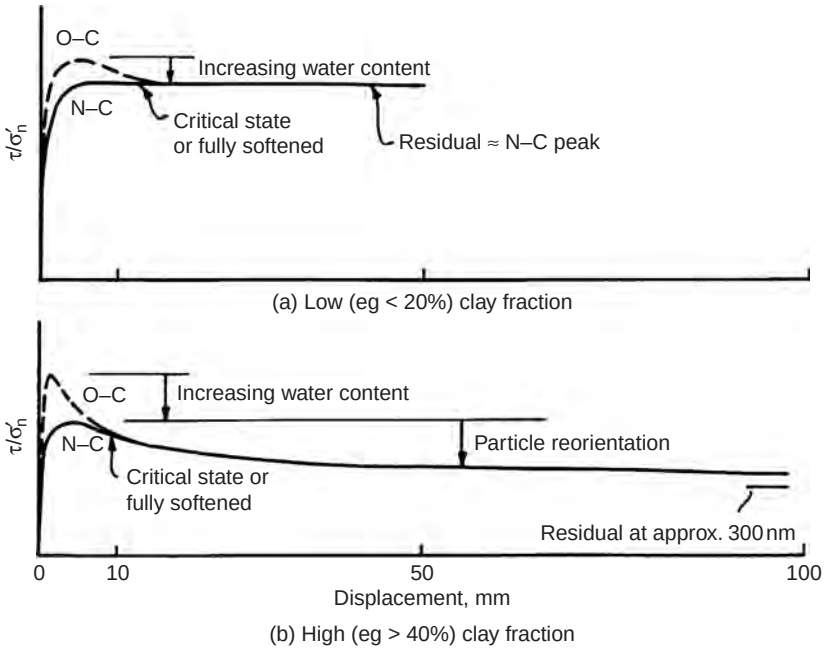


Figure 6.1 Diagrammatic stress displacement curves at constant normal stress σ'_n (Skempton, 1985).

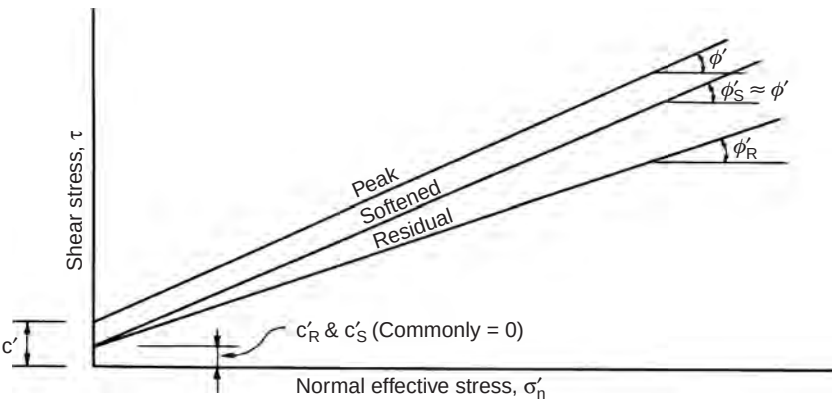


Figure 6.2 Relationship between peak, softened and residual strengths.

Section 6.1.4.4 this value differs from the laboratory residual strength depending on the testing method.

In practice, it may also reflect the roughness and waviness of the field failure surface compared to the planar surface in the laboratory sample.

An undisturbed sample of soil may behave in an overconsolidated manner at low normal stress and in a normally consolidated manner at high normal stresses (in excess

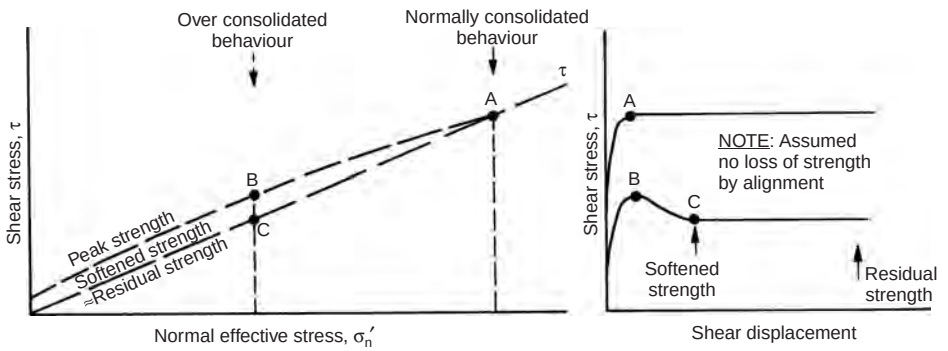


Figure 6.3 Diagrammatic stress displacement behaviour.

of the pre-consolidation pressure). This affects the pore pressure response of the soil during shear (see Section 6.1.2) and also the drained load-deformation behaviour as shown in Figure 6.3.

6.1.2 Development of drained residual strength ϕ'_R

There is a large amount of evidence that softening, with increased water content, and particle re-orientation, occurs on slide planes and that these lead to a reduction in shear strength from the peak strength. Skempton (1985), Lupini *et al.* (1981), Mesri and Cepeda-Diaz (1986) and Hawkins and Privett (1985) give good summaries of these effects.

Lupini *et al.* (1981) carried out a series of ring shear tests on sand-bentonite mixtures, and suggested that there were different mechanisms of residual plane development depending on the clay fraction percentage present. The mechanisms are turbulent or rolling shear, where the presence of rounded silt size particles prevents alignment of clay particles on the shear surface; sliding shear when the effects of the silt is overridden by the predominance of clay particles; and a transition between these two conditions. These results were summarized by Skempton (1985) and are reproduced in Figure 6.4.

Skempton (1985) presented results of field residual and ring shear tests on a range of soils. These are reproduced in Figure 6.5.

These figures show that 'sliding shear' with complete reorientation of the clay particles is likely to occur only where the clay fraction (finer than 0.002 mm) exceeds 50% of the total soil, and that for less than about 25% clay fraction, "turbulent" or "rolling" shear occurs without the influence of clay particle alignment. Note that Figure 6.5 only applies to clays with a Plasticity Index/Clay Fraction (PI/CF) ratio of 0.5 to 0.9.

It should be noted that the rather small range of residual strength for each clay size fraction in Figure 6.5 is an artefact of the few data points used in the plot. Figure 6.6 shows a plot with a much larger database. Some of the low clay size fraction data have very low residual friction angles. These are probably a result of polished surfaces

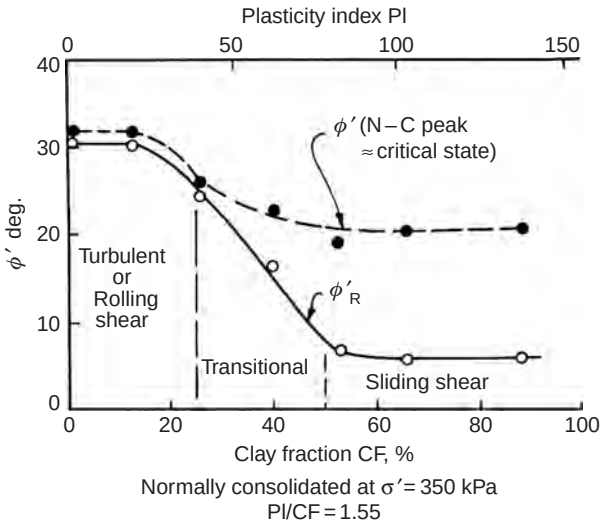


Figure 6.4 Ring shear tests on sand-bentonite mixtures (from Skempton 1985 after Lupini *et al.*, 1981).

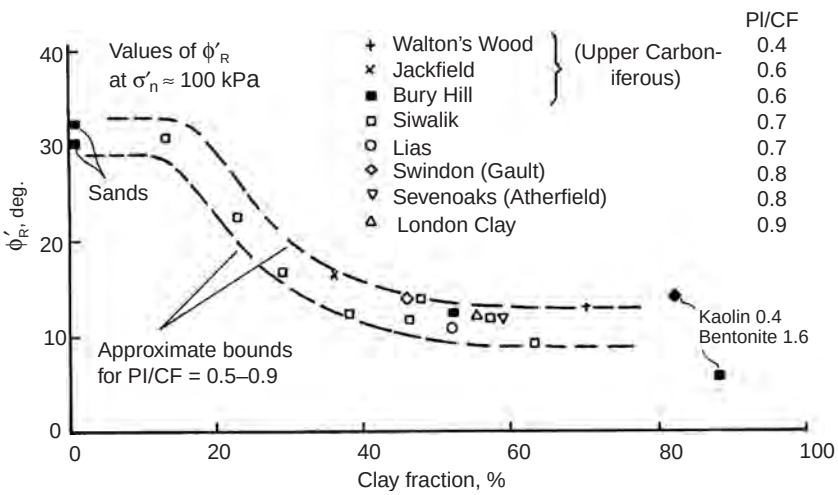


Figure 6.5 Field residual and ring shear tests on sands, kaolin and bentonite (Skempton, 1985).

resulting from tectonic shearing in the weak rock from which the samples were taken. The isolated high residual strengths at high clay size fraction are a result of the clay mineralogy as discussed below.

Apart from the clay fraction, the mineralogy of the clay also has an effect on residual strength. This is particularly so when the clay fraction is large. This reflects the fact that the different clay minerals have different particle shape and different

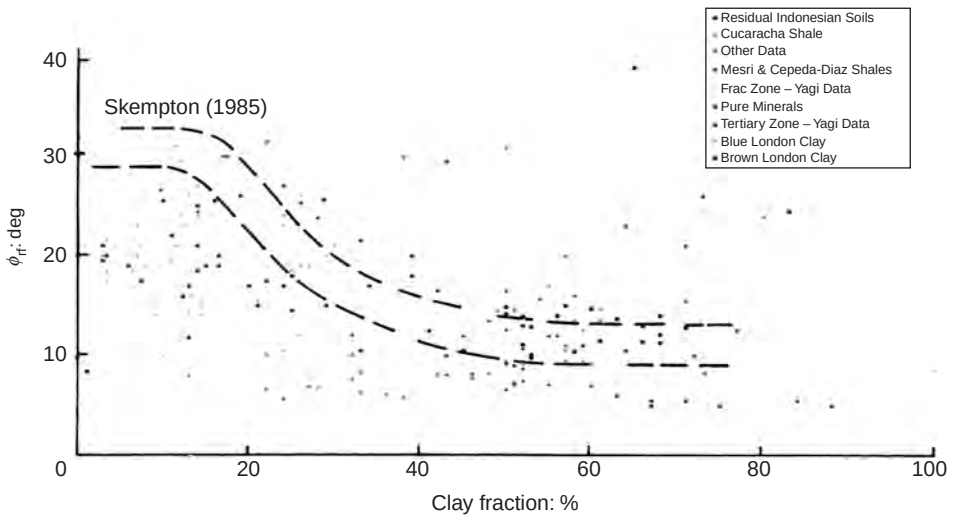


Figure 6.6 Laboratory residual strength versus clay size fraction (Lim, 2004).

inter-particle bonding. Most clay minerals, e.g. kaolinite, illite, chlorite and montmorillonite are platy structures, and are therefore subject to alignment when sheared. Montmorillonite has a particularly thin plate structure, and weak inter-plate bonds and leads to the lowest residual strength value with ϕ'_R as low as 5°. Kaolinite has a ϕ'_R of approximately 15° and illite approximately 10°.

Some clay minerals do not have a plate structure, e.g. halloysite has a tubular structure, attapulgite a needle-like structure and some amorphous clay sized minerals such as gibbsite, haematite, bauxite have essentially granular structures. This leads to much higher residual friction angles, commonly greater than 25° (Skempton, 1985).

The pore water chemistry may have an influence on the residual shear strength. Di Maio (1996a, b) showed that, with several clays from southern Italy and a commercial bentonite, there is a change in ϕ'_R from about 14° when prepared with saturated NaCl solution to about 6° when prepared with distilled water. Figure 6.7 shows this change for the bentonite. From a practical point of view, few natural soils are likely to be as affected as bentonite, but the residual strength parameters should be evaluated with pore water with chemistry close to the one existing *in situ* or anticipated during the life of the dam.

6.1.3 Undrained strength conditions

Undrained strength conditions occur when the loading time and the properties of the soil in the slope are such that there is insufficient time for pore pressures generated during the loading period to dissipate. Duncan (1996b) suggests this can be assessed from the dimensionless time factor, T which is expressed as:

$$T = \frac{C_v t}{D^2} \tag{6.1}$$

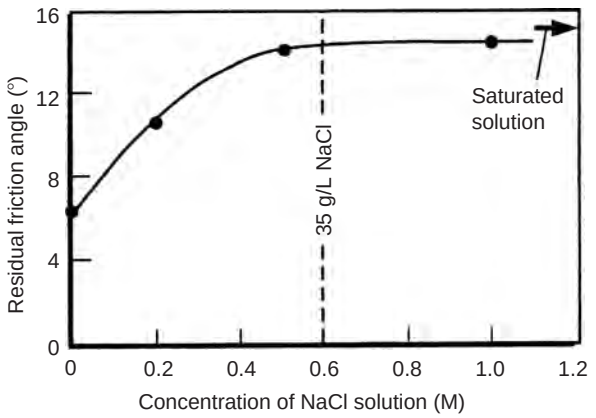


Figure 6.7 Residual friction angle of Ponza bentonite under normal stresses of 250–400 kPa for various concentrations of NaCl solution (Leroueil, 2000, after Di Maio, 1996b).

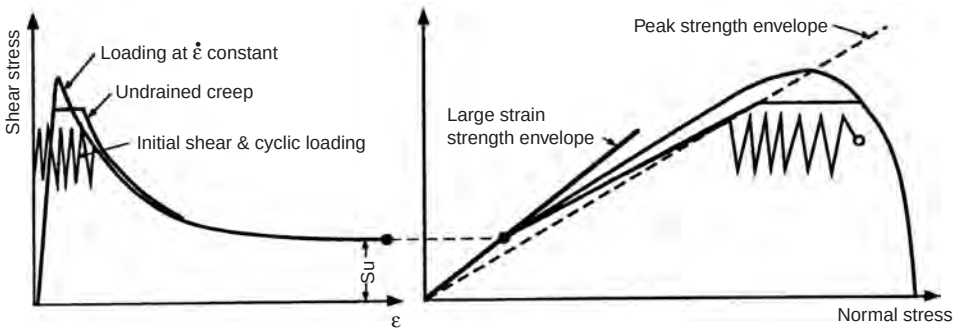


Figure 6.8 Typical undrained response of loose sands in monotonic, creep and cyclic loading (Leroueil *et al.*, 1996).

where C_v = coefficient of consolidation (m^2/year); t = loading time (years); D = length of the drainage path (metres).

Duncan indicates that, if the value of T exceeds 3.0, it is reasonable to treat the material as drained, and, if T is less than 0.01, undrained. If the value of T is between 3.0 and 0.01, both undrained and drained strengths should be considered. The undrained strength of a soil may be less than or greater than the drained strength depending on whether the soil is contractive or dilatent when it shears.

It is important to note that the strength available in the case of undrained shear of the contractive soil (S_u) is less than that for drained loading and less than assumed for conventional effective stress analysis, because of the positive pore pressures generated during shearing. As discussed below, this can happen in coarse grained as well as fine-grained soils.

Figure 6.8 shows stress strain and stress paths of saturated loose cohesionless sand loaded in triaxial compression under undrained conditions at a constant rate of

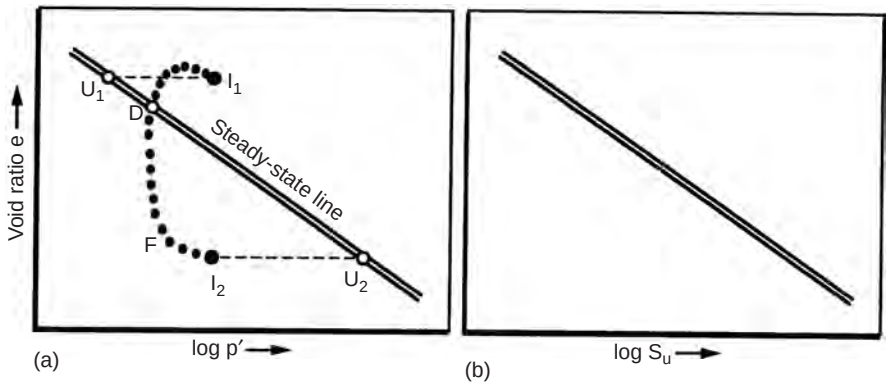


Figure 6.9 Steady state or critical state concept (Leroueil *et al.*, 1996).

strain, undrained creep, and an initial shear stress and cycle loading. In all cases the large strain or ultimate strength, also known as the steady state strength or residual undrained strength, is reached at large strains. In an e – log p′ diagram (Figure 6.9a), where e is the void ratio and p′ is the mean effective stress; such a test would have its initial point at I_1, and the ultimate critical or steady state at point U_1 on the steady state line. For initial condition at point I_2, below the steady state line, the undrained behaviour would be dilatant with a steady state at point U_2.

Whether a soil will be contractive or dilative depends on its initial state condition, i.e. whether it is above or below the steady state line, but also the stress path. This is discussed further in Leroueil (2000) and Fell *et al.* (2000).

In some contractive soils, e.g. loose sands, loaded in saturated, undrained conditions, the triaxial compression stress path is more complex, as shown in Figure 6.10. The soil reaches a peak undrained strength at stresses less than the peak strength (at A).

The line through these points is termed the collapse surface (Sladen *et al.*, 1985; Dawson *et al.*, 1998), or critical stress ratio (Vaid and Eliadorani, 1998) or the instability line (Lade, 1992). At these stress conditions the soil wishes to contract as it shears but, being constrained by the no volume change and conditions of saturated undrained conditions, develops positive pore pressures resulting in weakening to the ultimate strength Su at B.

The important point here is that the collapse surface strength may be lower than the peak, or critical state (constant volume) strength, which is commonly used in stability analysis, so failure may occur at stresses lower than expected (e.g. Dawson *et al.*, 1998). As pointed out by Sladen *et al.* (1985), for very loose sands whose state lies on the collapse surface, only minimal excess pore pressure is necessary to trigger collapse. Loading may be essentially drained up to the point of reaching the collapse surface.

The degree of strain weakening has been characterized in terms of the Brittleness Index I_B (Bishop 1971):

$$I_B = \frac{\tau_p - \tau_r}{\tau_p} \times 100\% \tag{6.2}$$

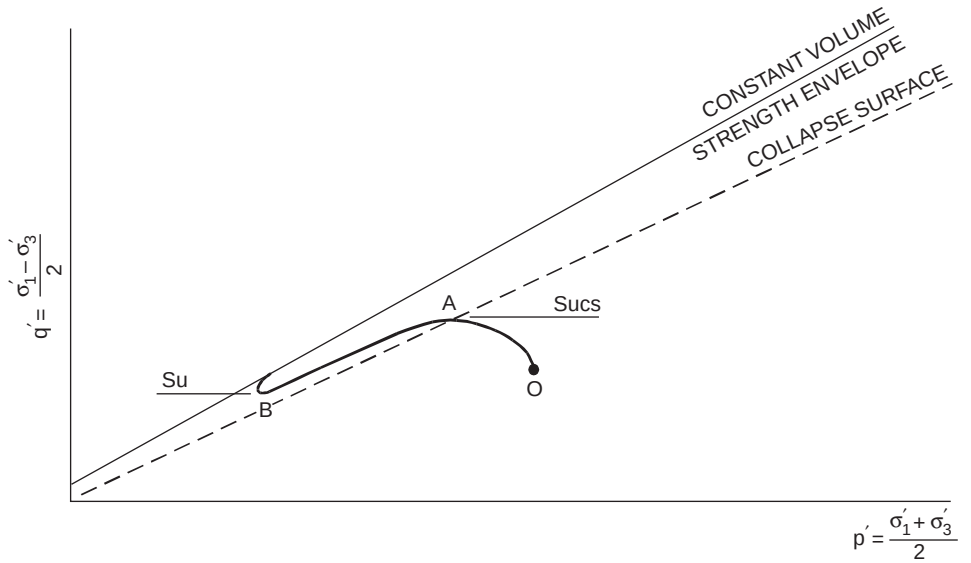


Figure 6.10 Stress path for saturated loose sand which displays collapse behaviour.

where τ_p and τ_r are the peak and residual strengths defined under the same effective normal stress.

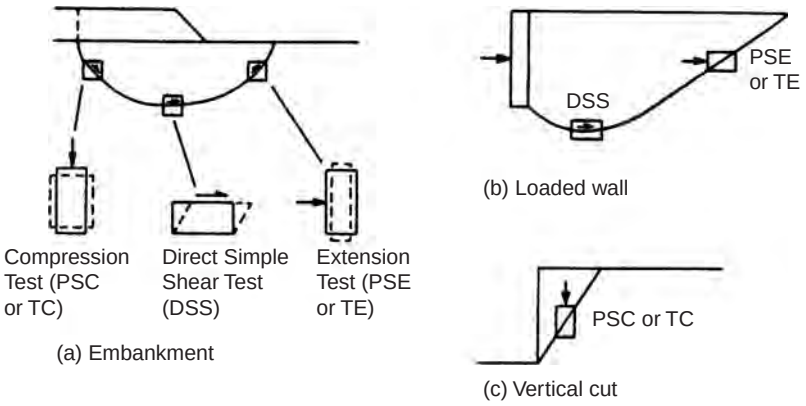
However, as indicated by Vaughan & Hamza (1977) and by Chandler (1984) the brittleness index alone is not sufficient to characterise the susceptibility of a soil to progressive failure and the rate at which the strength decreases from peak strength to ultimate strength is also important. D'Elia *et al.* (1998) propose a generalised Brittleness index, I_{BG}, defined as follows:

$$I_{BG} = \frac{\tau_p - \tau_{mob}}{\tau_P} \times 100\% \tag{6.3}$$

in which τ_{mob} is the mobilised shear stress at the considered strain or displacement.

I_{BG} thus varies with strain or displacement from 0 at the peak to a value equal to I_B at large displacements. I_{BG}, is associated to the stress paths that are representative of those followed in situ, and must thus not be seen as a fundamental characteristic of a soil. With this extended definition, not only overconsolidated clays, clay shales, sensitive clays, residual soils and loess, but also cohesionless soils such as loose sands, may appear brittle in undrained conditions.

Contractant conditions may occur in mine tailings, end-tipped mine waste dumps, uncompacted or poorly compacted road or railway fills, normally and lightly over-consolidated clays, poorly compacted clay fills (e.g. old dams which were compacted by horses hooves or rolled with light rollers in thick layers), puddle cores, and in the core of large dams where the confining stresses exceed the pre-consolidation effects of compaction, and potentially in loose dumped dirty rockfill in dams (although the authors are not aware of this happening).



Note: Plane strain tests (PSC/PSE) used for long features
Triaxial tests (TC/TE) used for near symmetrical features
Direct shear (DS) normally substituted for DSS to evaluate $\bar{\phi}$

Figure 6.11 Relevance of Laboratory Shear Tests to Shear Strength in the Field (Kulhawy, 1992).

Cooper *et al.* (1997) discuss an example where the use of conventional effective stress analysis over-estimated by about 50% the factor of safety of an earth dam constructed with poorly compacted clay and with concrete core wall.

As has been recognized for a long time, e.g. Jamiolkowski *et al.* (1985), Ladd (1991) and Kulhawy (1992), the undrained shear strength used in the analysis should take account of the failure mechanism. As shown in Figure 6.11 the loading conditions under an embankment are best simulated by triaxial compression, while beyond the toe, they are best simulated by triaxial extension and direct simple shear. Centrally within an embankment they are probably represented by triaxial compression, and at the upstream and downstream toe by triaxial extension.

Figure 6.12 shows the mean normalized undrained strength ratios for the major laboratory tests.

For this figure the reference strength ratio is given by the modified Cam Clay model, which Kulhawy (1992) indicates is reasonable for relatively unstructured soils. For sensitive, cemented and other structured fine grained soils, the reference strength is a lower bound. It can be seen that to use Ko triaxial compression tests is un-conservative for those parts of the failure surface best modelled by triaxial extension and direct simple shear tests.

The behaviour of contractant granular soils is a complex, developing science, and will not be further discussed here. Reference should be made to Fell *et al.* (2000) and the references given therein and to the recent literature.

6.1.4 Laboratory testing for drained strength parameters, and common errors

The following discussion on shear strength of soils briefly outlines the test procedures and some of the problems which arise in testing and interpretation of results. It is

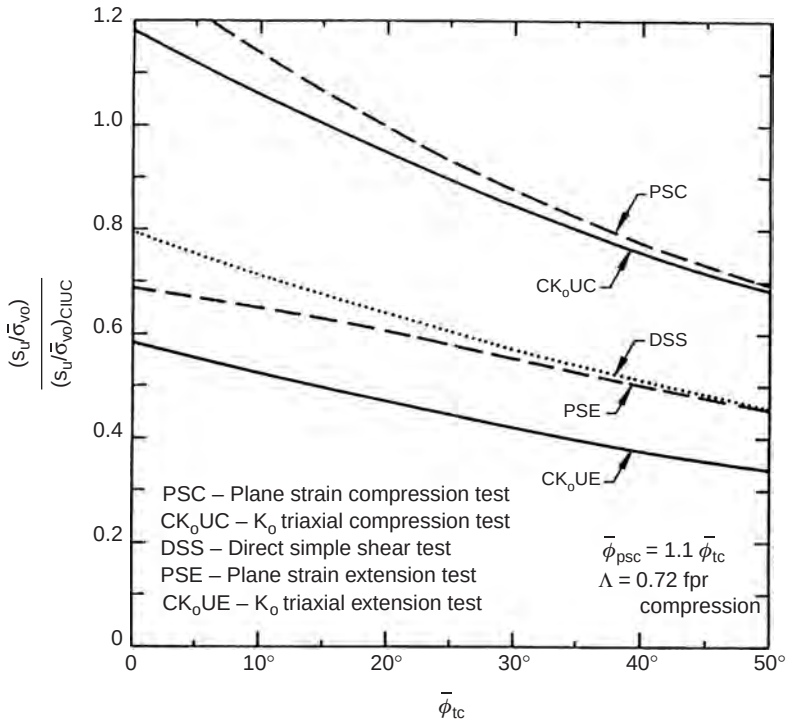


Figure 6.12 Mean Normalised Undrained Strength Ratio for Major Laboratory Tests (Kulhawy, 1992).

the authors' experience that many organisations, at least in the past, failed to follow correct test procedures despite the fact that they have been long established.

6.1.4.1 *Triaxial test*

Triaxial tests are the most common means of obtaining the peak shear strength parameters c', ϕ' and, provided constant volume conditions are reached within the strain capability of the test, the fully softened strength. The triaxial test cannot be used to obtain residual strengths because it is not possible to subject the soil to sufficient displacement on the shear surface to reach these values.

Most triaxial testing is triaxial compression, carried out at a nominally constant rate of axial strain. Tests are usually saturated consolidated undrained with pore pressure measurement (CUDPP) or saturated consolidated drained (CD). For practical purposes these yield the same strength (Duncan, 1994) provided the tests are performed correctly. Most triaxial testing is CUDPP because the time for testing is less than for CD. However, in testing some soils, the pore pressure response during testing is such that the CUDPP circles fall close together and CD tests may be needed to obtain a wide enough range of Mohr circle plots to draw an accurate envelope. Staged testing, where one sample is tested for three consolidating stresses, is sometimes used because it involves less sample preparation, but it can lead to errors as described below.

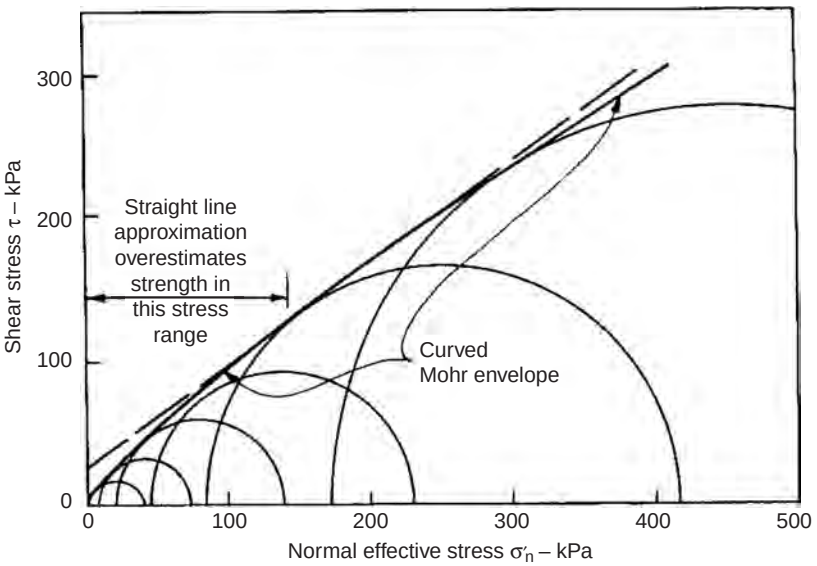


Figure 6.13 Typical Mohr's envelope plot.

Details of testing equipment and test procedures are given in Head (1985), Bowles (1978) and Bishop and Henkel (1971) and the relevant ASTM (D2850-95, D4767-95), British (BS1377-8:1990) or other standards (e.g. Australian Standard AS1289 6.4.1 and 6.4.2, 1998). Lade (1986), Saada and Townsend (1981) and Baldi *et al.* (1988), Germaine and Ladd (1988), Lacasse and Berre 1988) give summaries of triaxial testing.

There are several common sources of error in triaxial testing, which often lead to an overestimation of effective cohesion, usually with a reduction of effective friction angle. This may lead to an overestimation of strength in the working stress range. The sources of error are:

(i) *Testing at a too high stress range.* The Mohr's circle envelope for most over consolidated soils is curved in the manner shown in Figure 6.13. If laboratory tests are carried out at higher stresses than will be encountered in the field, this will result in an overestimation of shear strength in the field. If the field stresses were between 0 and 100 kPa and testing had been carried out at confining stresses between 100 and 400 kPa, the strengths would be overestimated by 10% to 300%. This potential problem can be overcome by specifying the correct stress range for the triaxial tests and by recognising possible curvature effects when selecting design parameters from the p′-q plot. It is also important to plot thed individual Mohr's circle plots for a sample, so the curvature can be identified.

(ii) *Not saturating the sample adequately.* Soil samples (even if taken from below the water table) may not be saturated before testing in the laboratory because:

- Fissures in the soil open up due to sampling and unloading.
- Pore pressure redistributes in non homogeneous soils on unloading.

- Air intrusion may occur at the surface of the sample.
- Dissolved gas may come out of solution.
- Air may be trapped between the sample and the membrane in the triaxial test.
- Compacted soils even if compacted wet of optimum water content, as is commonly required in dam construction, are usually only 90% to 95% saturated.

The effects of partial saturation are:

- Pore pressure changes during shearing are less than for a saturated sample. However, the trend (whether +ve or −ve changes) is not affected (Lade, 1986).
- Because pore pressure changes are less than for a saturated soil, the undrained strength will be affected. For over-consolidated clays, Δu is −ve, and lower than for a saturated soil, so the undrained strength is reduced; for normally consolidated clays, Δu is +ve and lower than for a saturated soil, so the undrained strength is increased.
- The magnitude of actually measured pore pressures may be incorrect, usually underestimated, because of the presence of air in the pore system.

Provided the pore pressures are measured correctly, the effective strength envelope will not be greatly affected even though the total stress envelope is markedly affected.

To avoid problems of partial saturation, the laboratory samples should be saturated by percolation followed by back pressure saturation. Percolation by itself will not achieve saturation even in permeable soils (e.g. sands). Back pressure saturation is needed to reduce the volume of the air bubbles in the pore water (Boyle's Law) and to drive air into solution (Henry's Law). The degree of saturation should be checked by monitoring the increase in pore pressure for an increment in cell pressure in undrained conditions:

$$\Delta u = B\Delta\sigma_3 \tag{6.4}$$

where B is defined by Skempton (1954) as:

$$B = \frac{1}{1 + \frac{nC_v}{C_{sk}}} \tag{6.5}$$

where n = porosity; C_v = compressibility of pore fluid; C_{sk} = compressibility of the soil skeleton. For a saturated soil $C_v = C_{water}$ and C_v/C_{sk} is very small, hence B = 1. However, for very stiff soils, and very weak rocks, C_{sk} is also small and it is difficult to achieve B = 1 as shown in Figure 6.14. In these circumstances it is sufficient to check B for several increments of cell pressure and, provided B is constant, the correct effective strength parameters will be obtained (Lade 1986). For very stiff soils and very weak rocks, the stiffness of the testing system becomes significant and may lead to low measured B values.

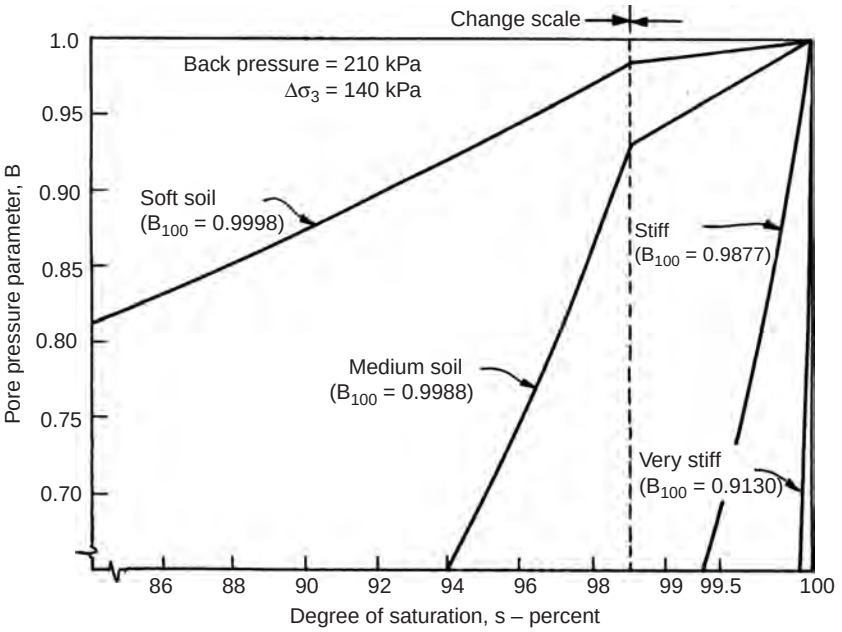


Figure 6.14 Variation of B-value with degree of saturation for four classes of soil (Black and Lee, 1973).

Typically back pressure saturation will be carried out at 300 kPa to 1000 kPa, but may be as low as 200 kPa for soft soils. It should be noted that dissolving the air in the water takes time (often many hours). Black and Lee (1973) give a method to calculate this time.

(iii) *Testing at too high a strain rate*. CD and CUDPP triaxial tests must be sheared at a sufficiently slow rate to allow dissipation of pore pressure in the CD test and equalisation of pore pressure throughout the sample in the CUDPP test.

The effect of testing at too high a strain is illustrated in Figure 6.15 for an overconsolidated clay.

If a CUDPP test is sheared too quickly, the pore pressure changes measured at the ends of the sample are less than the actual changes at the centre of the sample where shearing is occurring, because of the restraint effects of the top and bottom of the sample. Since there is a reduction in pore pressure during shearing of an overconsolidated clay, the Mohr circle plots based on measured pore pressures are displaced to the left of where they should be.

The negative pore pressure response to shearing is less for the sample at higher confining stress where the soil behaves closer to a normally consolidated soil, so the displacement of the Mohr's circle for the higher confining stress is less than that for the lower confining stress.

This yields overall an increase in c', and a decrease in ϕ'. The effect is similar for CD tests, where negative pore pressure will remain in the centre of the sample if the strain rate is too high, giving an apparent increase in failure stress

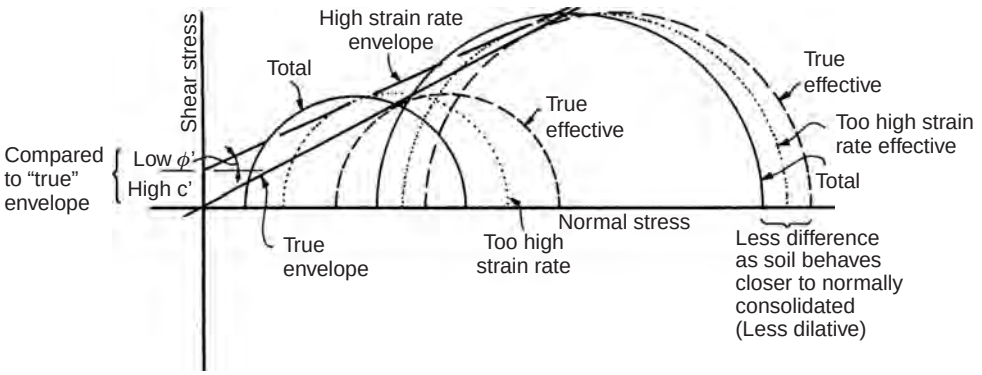


Figure 6.15 The effect of high strain rate on CUDPP triaxial tests.

(σ'_1), which again will be greater for samples at lower confining stresses than for higher confining stresses.

It is generally accepted (e.g. Germaine and Ladd, 1988; Bishop and Henkel, 1957, 1971; Lade, 1986) that 95% dissipation (for drained tests) or 95% equalisation (for CUDPP tests) gives adequately accurate results.

Bishop and Henkel (1957, 1971) used consolidation theory to develop the following approximate equations to estimate the time for failure:

Drained tests (CD)

$$t_f = 6.7H^2/Ca \tag{6.6}$$

for top and bottom drainage:

$$t_f = 0.5H^2/Ca \tag{6.7}$$

for top, bottom and side drainage

Undrained tests with pore pressure measurement (CUDPP)

$$t_f = 1.7H^2/Ca \tag{6.8}$$

without drainage

$$t_f = 0.07H^2/Ca \tag{6.9}$$

with top, bottom and side drainage.

In these equations t_f = time for failure; H = half height of sample = diameter; Ca = coefficient of consolidation

To determine the strain rate, the strains to failure must be estimated. This may be up to 20% for normally consolidated soils, and only 1 to 2% for heavily overconsolidated soils.

Ca should be determined from the consolidation phase of the test, but it should be noted it will vary with degree of over consolidation (Germaine and

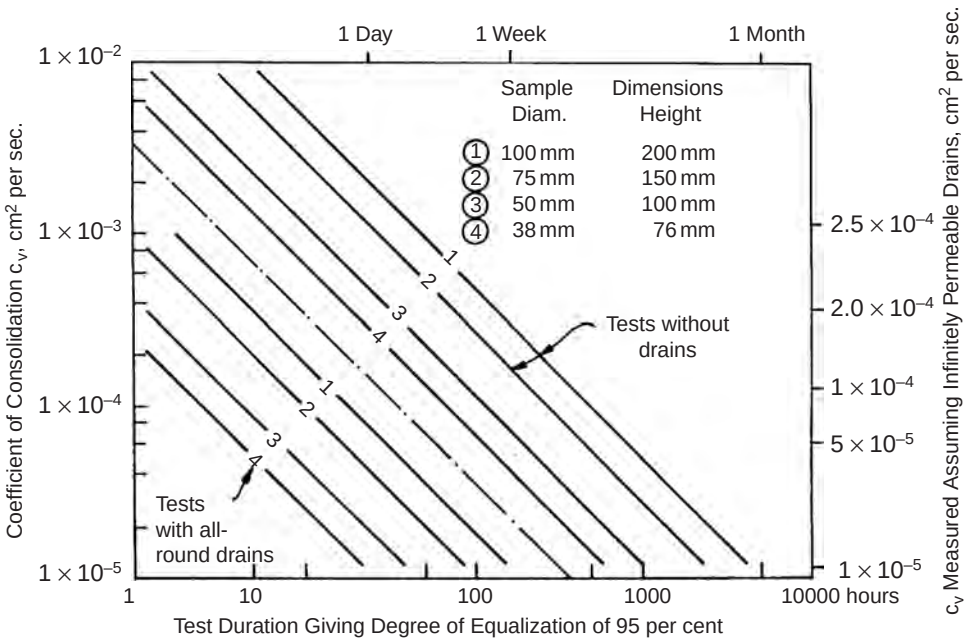


Figure 6.16 Chart for finding durations of drained and undrained tests for 95% dissipation at failure (Blight, 1963).

Ladd, 1988; Leroueil *et al.*, 1988), so should be measured at the end of the consolidation.

Blight (1963), Lade (1986), Germaine and Ladd (1988), Leroueil *et al.* (1988) all discuss the problem of efficiency of the side drains (usually strips of filter paper placed between the soil sample and the rubber membrane in the triaxial test). Leroueil *et al.* (1988) point out that, in the over-consolidated range, the drains are usually able to cope with the required water flow, but in the normally consolidated range, they may not have adequate discharge capacity. Germaine and Ladd (1988) indicate this is a problem for soils with a permeability greater than 10^{-10} m/sec (i.e. virtually all soils) Lade (1986), recommends the use of Figure 6.16 based on the theory of Bishop and Henkel and experimental results.

Akroyd (1957, 1975), Head (1985) and Bowles (1978) all give relationships for estimating time for failure based on monitoring the consolidation stage of the test.

Lade (1986) and Germaine and Ladd (1988) indicate that, by using lubricated end plates, pore pressures are more uniform and CUDPP tests can be carried out more quickly. However no guidelines are given to determine an acceptable strain rate. Lubricated end plates do not reduce the required time for failure in CD tests since pore pressure dissipation, not equalisation, is required.

(iv) *Staged testing*. In staged testing, the one soil sample is usually saturated, consolidated and sheared at the lowest selected confining stress, then consolidated to the second confining stress and sheared, then further consolidated to the

third confining stress and sheared. This reduces the amount of sample preparation time, and hence the cost of testing. The procedure often gives acceptable results, but may tend to give lower strengths for the second and, particularly third stage, when testing strain weakening or compacted dilative soils due to sample dilation and softening, and loss of strength due to displacement on the shear plane. Where staged testing gives high effective cohesions, the third (and second) stage testing should be checked with tests on samples consolidated to the third stage confining pressure without earlier staged testing. The authors' recent experience is that staged testing almost always gives excessively high c' for compacted soils, and recommend it not be used. Staged testing should not be used for sensitive or cemented soils.

The shear strength c', ϕ' of a compacted earthfill will be somewhat dependent on the degree of compaction, with higher compaction, leading to minor increase in c' and ϕ'. When testing earthfill for dam construction, the laboratory tests should be carried out on samples compacted at compaction density and moisture content consistent with what will be specified for the dam construction. Where there are fabric features present (e.g. fissures), staged testing is desirable so the complete test is carried out on the one feature.

6.1.4.2 *Direct shear test*

Direct shear tests are used to obtain the peak and fully softened or critical state (constant volume) strengths of granular soils (sand, silt, gravel) and cohesive soils. They are also used to determine the residual strength of cohesive soils, by using the multiple reversal technique which is described below, and the strength on existing surfaces of weakness in soil, e.g. shear surfaces, slide surfaces and fissures.

These tests are all done in drained conditions. Those for cohesive soils are saturated prior to testing. The test is seldom used to determine undrained strengths and in the opinion of the authors should not be.

Details of testing equipment and test procedures are given in Head (1980, 1981, 1985), ASTM D3080-98, BS1377:1990 and AS1289, 6.2.2-1998.

It is the authors' experience that when direct shear and triaxial test data are available, the peak strengths derived from direct shear tests are sometimes higher than from the triaxial tests. This is mainly due to the inability to saturate the soil properly, along with the problem that the direct shear forces failure on a pre-determined plane, which is not coincident with the principal stress direction at failure (Saada and Townsend, 1981).This means that care should be taken in using the direct shear results and the test should not be used as the primary means to obtain peak strengths for cohesive soils. The direct shear is a good way to obtain the residual strength.

True laboratory residual strength of clay soils only develops with significant displacement. Skempton (1985) gives the "typical displacement values" reproduced in Table 6.1.

Most laboratory shear box equipment is 75 mm × 75 mm or 60 mm × 60 mm, and the maximum practical displacement is of the order of 6 mm to 10 mm, sufficient only to measure the peak and possibly the softened strength.

Residual strength is therefore obtained with a direct shear machine by repeatedly shearing the sample until the strength is not further reduced by further shearing (the

Table 6.1 Typical displacements at various stages of shear in clays having clay fraction[3] >30% (Skempton, 1985).

	Displacement mm[2]	
Stage	*Overconsolidated*	*Normally consolidated*
Peak	0.5–3	3–6
Rate of volume change approx zero[1]	4–10	
At residual + 1° ($\phi'_R + 1°$)	30–200	
At residual (ϕ'_R)	100–500	

Notes: (1) i.e. fully softened strength.
(2) For $\sigma'_n < 600$ kPa.
(3) Clay fraction = % finer than 0.002 mm.

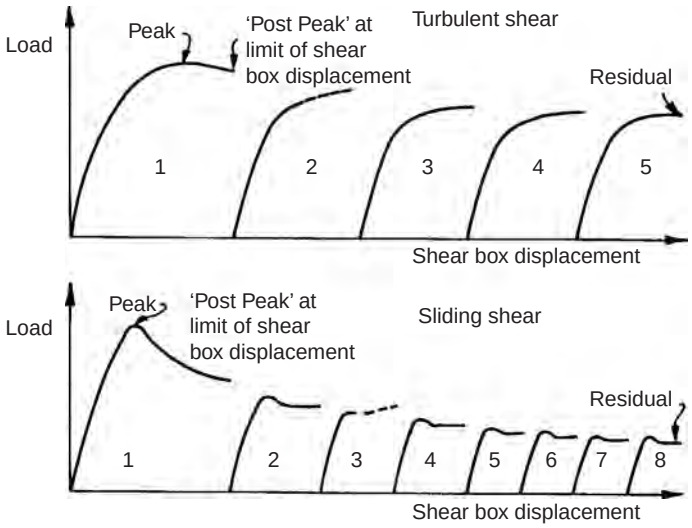


Figure 6.17 Typical load displacement curves for direct shear tests (Moon, 1984).

'multiple reversal' method). Typical load displacement curves for 'turbulent' and 'sliding' shear are shown in Figure 6.17 for tests which are progressing well. Most tests are more irregular.

The repeated shearing is achieved in several ways:

- Shear to limit of travel, unload (partly), rewind rapidly to start, then reload and re-shear etc. until a minimum is reached, or
- Shear to peak, then unload partly, and wind back and forth rapidly until a total travel of say 50 mm is achieved, reload and re-shear etc. until a minimum is achieved, or
- Shear back and forth under load at a slow strain rate until a minimum is achieved.

Further acceleration of achievement of the residual strength is sometimes accomplished by cutting a slide plane into the sample prior to testing. The cut surfaces are sometimes polished on a glass plate prior to testing.

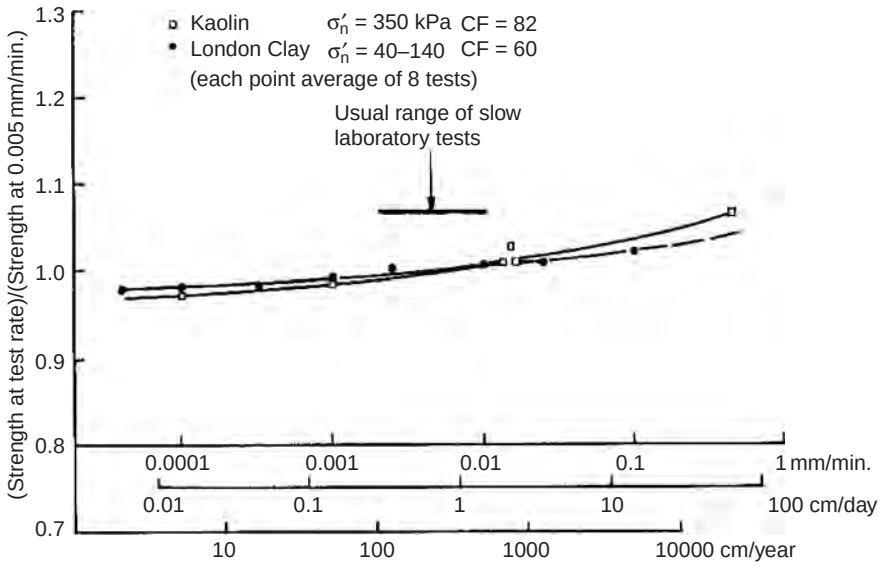


Figure 6.18 Variation in residual strength of clays at slow rates of displacement (Skempton, 1985).

Problems which arise in the use of the direct shear to assess peak strength include:

- It is difficult to ensure that the sample is properly saturated. Saturation is achieved by soaking under a confining stress until there is no further volume change. Unlike triaxial testing, pore pressure cannot be measured, so it is necessary to rely on monitoring the consolidation deformation of the sample during saturation and consolidation. The best approach is to saturate under a low confining pressure until swell (or consolidation) ceases, then load to the first test confining pressure, and monitor until settlement ceases.
- Testing at a too high strain rate leads to overestimation of shear strength in a similar way to triaxial testing, i.e. c' is overestimated, ϕ' underestimated. This can be overcome by monitoring the consolidation to obtain t_{50}, t_{90} or C_v value, and then using formulae developed by Akroyd (1975), Bowles (1978) or Head (1985) to estimate the strain rate. Skempton (1985) showed however that the residual strength is not greatly sensitive to strain rate provided reasonably low strain rates are used. This fact may be used to speed up test rates, provided that when a constant shear stress has been reached the strain rate is reduced, and the resulting shear stress is not significantly lower than the previous minimum (see Figure 6.18).
- Testing at a too high confining stress range, and staged testing can lead to an overestimation of c', underestimation of ϕ' in the same way as for triaxial testing.

Problems which arise in the use of the direct shear to assess residual strength include:

- Repeated reversal of the direction of shearing is necessary, and this may partially destroy the alignment of particles on the shear plane, preventing a true residual strength being achieved.
- Repeated reversal of the shear box often leads to soil squeezing out between the two halves of the box. If this happens, or if the box tilts, interference can occur between the two halves of the box leading to erroneous results.

The authors' experience is that one of the biggest problems is that laboratory personnel (and the engineers supervising them) do not understand the definition of residual strength. Figure 6.19 shows some of the all too common misconceptions.

6.1.4.3 *Ring shear test*

To overcome the problems of multiple reversals of a shear box and to obtain sufficient displacement on the slide plane to achieve residual strength, Bishop *et al.* (1971) developed a ring shear apparatus.

In the ring shear, an annular ring shaped specimen (Figure 6.20) is subject to a constant normal stress σ'_n confined laterally, and caused to shear on a plane of relative rotary motion at a constant rate of rotation.

For the Bishop *et al.* (1971) machine (manufactured by Wykeham Farrance) the sample is 150 mm outside diameter, 100 mm inside diameter, and 19 mm thickness. Remoulded samples are usually tested, but undisturbed samples can be placed in the equipment. The machine is mechanically complex and expensive, putting it largely in the research category. Bromhead (1979) and Bromhead and Curtis (1983) describe a simpler ring shear device. The test procedure is detailed in BS1377-7:1990, and ASTM D6467-99. Bromhead (1986) and Clayton *et al.* (1995) give some practical details on how to operate the equipment. The sample in the Bromhead machine is outside diameter 100 mm, inside diameter 70 mm and only 5 mm thick.

The ring shear devices have essentially unlimited "strain" availability, so overcoming the major objection to the reversing direct shear test. However, there are limitations:

- only remoulded samples can be tested (in the Bromhead machine).
- only the residual strength can be obtained. The peak strength is affected by remoulding and non-uniform stress conditions.
- the sample may tend to squeeze out the sides of the ring. The Bishop machine is constructed to overcome this and can be used for softer clays than the Bromhead machine.
- saturation and testing at a strain rate appropriate to give the required drained conditions is necessary as for the direct shear test.

6.1.4.4 *Comparison of field residual with laboratory residual strength obtained from direct shear and ring shear*

There have been a number of papers published comparing field residual strengths obtained by back-analysis of landslides, and laboratory direct shear testing of the

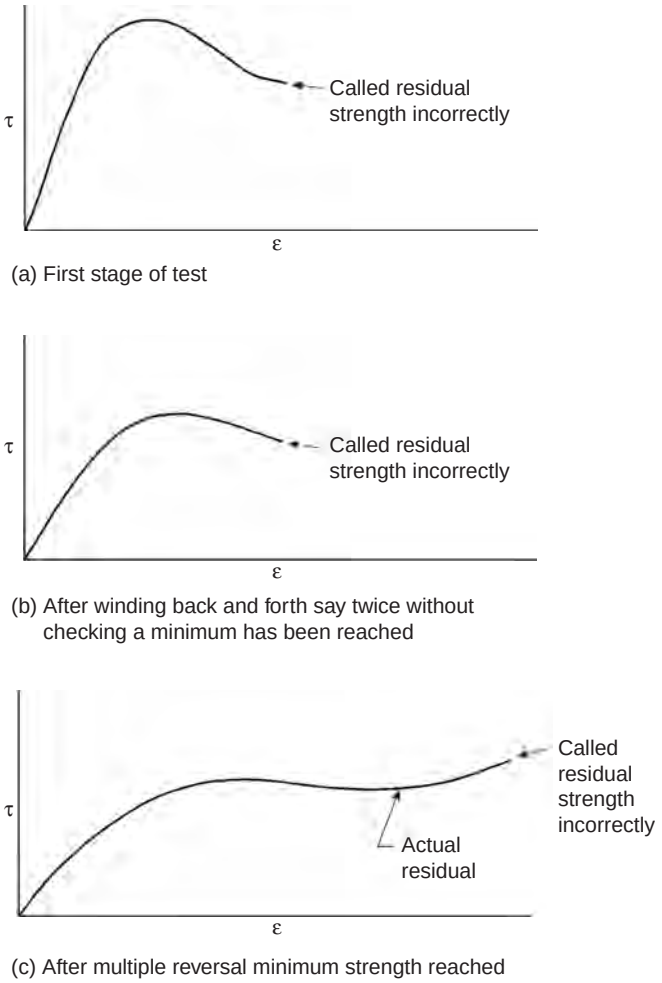


Figure 6.19 Some all too common misconceptions in interpreting residual strength from direct shear tests.

undisturbed slide plane, with residual strengths obtained from multiple reversal direct shear, and ring shear tests. These include Bishop *et al.* (1971), Townsend and Gilbert (1973), Bromhead (1979) and Bromhead and Curtis (1983) and Skempton (1985).

In summary the literature indicates that:

- Direct shear tests on the slide plane or bedding plane shear are the most reliable indicator of field residual strength.
- Ring shear devices either underestimate by 1° to 2° or approximate the field residual.
- Multiple reversal direct shear on clays will probably overestimate field residual by 1° or 2°.

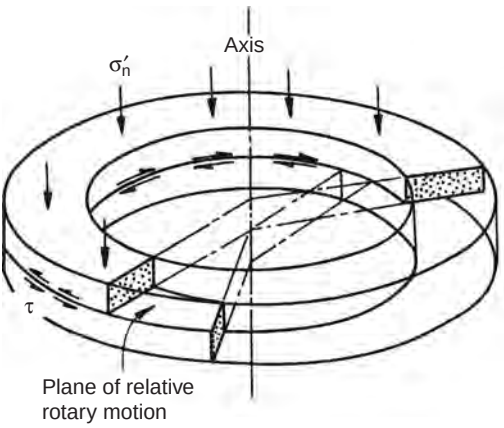


Figure 6.20 Ring shear test sample.

– The difference between multiple reversal direct shear and field residual is only small in weak rocks such as shales and claystone.

The authors' experience in testing clays and very weak claystone is that the difference may up to 6°. Hence direct shear tests on the slide surface or bedding surface shear should always be done if practicable.

The authors' experience in testing soil from several landslides where soils were derived from sedimentary rocks, tuff and basalt is that the Bromhead ring shear is very susceptible to the presence of particles of sand. These 'catch' on the shear surface and lead to higher residual strengths than obtained from multiple reversal direct shear or tests on slide planes. Comparable results were only obtained by drying and sieving the soil on a 0.150 mm sieve to remove any particles of sand before re-wetting it to the plastic limit for testing.

It can be seen that ideally tests should be carried out on the low strength planes if already present, or that ring shear tests should be carried out provided that the soil is free of sand particles. Unfortunately, ring shear machines are not widely available and reversing direct shear tests have to be used. In these cases, the design strength should be selected at the lower bound of results unless exhaustive testing shows this to be conservative. This is valid since few (if any) of the reversing tests may have achieved a proper alignment of particles, and the low results (provided these were not testing errors) are probably the best indicator of residual strength. The validity of the results should also be checked by comparison with published data, e.g. that in Figure 6.5. The notable exceptions to the data on that figure are allophane and halloysite clays which do not have classical platy structure, so give higher values (e.g. $\phi'_R = 25°$ to 35°).

In many cases of slope instability, where stabilization measures are being designed for an existing slide feature, the strength can be determined by back analysis and stabilization works designed on the basis of increasing the factor of safety by no less than 30%. This obviates some of the need for accurate knowledge of the shear strength from laboratory tests and allows for such factors as waviness on the shear surface.

However care must be taken in back-analysis because if pore pressures are over-estimated this will result in over-estimation of the shear strength.

A method which has been used by the authors to determine the "lower bound" residual strength of a soil, is to separate the clay fraction from the silt (and sand) of the soil by sedimentation. The clay fraction is then tested by overconsolidating it in the direct shear machine, and then determining the residual strength by multiple reversal direct shear testing. This procedure was suggested by Pells (1982) and has given reasonable values when used. More recently the ring shear has been used to test the minus 0.150 mm sieve soil as described above.

In nature, it is not uncommon to observe a clay rich zone 0.2 mm to 1 mm thick at the slide surface, and in these circumstances the procedures outlined above can be expected to give a good representation of the field condition.

6.1.5 Laboratory testing for undrained strength

The undrained strength of cohesive soils can be obtained from several laboratory tests:

(a) Unconfined compression (UC)
(b) Unconsolidated undrained (UU)
(c) Consolidated undrained (CU)

It must be recognised (Germaine and Ladd, 1988; Jamiolkowski *et al.*, 1985) that the undrained strength is affected by:

- The degree of disturbance of the soil sample. Disturbance can occur because of stress relief on sampling; the sampling technique (disturbance as the thin wall tube or other sample is pushed into the soil); handling (e.g. drying or disturbance during transportation, extrusion of the sample from the sampling tube, or trimming in the laboratory).
- The water content of testing, e.g. soils allowed to dry from their field water content will be stronger than those tested at the field water content. Thin-walled tubes should be sealed as soon as they are taken to avoid drying.
- The stress path, in particular whether tested in compression, extension, or direct simple shear (see Figures 6.11).
- The test procedure, e.g. UC and UU tests do not simulate the *in situ* test stresses.

Sample disturbance for soft clays, such as may be present in foundations of small dams, puddle cores, or in saturated, poorly compacted clay fill; and the sensitivity of undrained strength to the water content for stiff clays, make UC and UU tests totally unreliable, and they should not be used for design purposes. In particular for soft clays they may seriously underestimate the real strength – but you never know by how much.

Tests should therefore be done by CU methods, and they should, so far as practicable, simulate the *in situ* stress conditions, and the stress path.

For testing of soft clays, the most commonly used methods are the recompression test, and the SHANSEP technique. These are discussed in Jamiolkowski *et al.* (1985), Germaine and Ladd (1988) and Lacasse and Berre (1988).

The recompression technique was developed by the Norwegian Geotechnical Institute (NGI), and is described in Bjerrum (1973) and Jamiolkowski *et al.* (1985). Samples are consolidated to the same effective stress condition as experienced *in situ*, before being sheared undrained, either in compression or extension. That is, the samples are consolidated under σ'_{vo} and σ'_{ho}. Ideally a constant Ko($= \sigma'_{ho}/\sigma'_{vo}$) is maintained during consolidation, but NGI has shown that the simpler procedure of consolidating isotropically under σ'_{ho}, then increasing σ'_{v} to σ'_{vo} and consolidating further, gives satisfactory results. Lacasse and Berre (1988) indicate that for soils with an OCR less than 1.5, they recommend a two stage application of the cell pressure, and application of the deviatoric stress in 4 to 6 steps. For other soils, a single step consolidation is acceptable.

The SHANSEP – "Stress History And Normalised Engineering Properties" – technique was developed by Ladd and others. It is described in Ladd and Foot (1974), Ladd *et al.* (1977) and Jamiolkowski *et al.* (1985).

Note that for both SHANSEP and the recompression techniques, Su should be determined in compression or extension, whichever is appropriate. Both techniques are claimed to give high quality strength data, which compares well with strengths determined by back analysis of failures. The recompression technique is simpler to use, requiring less laboratory testing, but can give an overestimate of strength in normally and lightly over-consolidated clays. SHANSEP involves consolidation of samples beyond their *in situ* stresses, and may lead to destruction of soil structure and cementing, giving an underestimate of strength.

In many projects the SHANSEP technique would be too expensive, particularly since a S_u/σ'_{vo} vs Over Consolidated Ratio (OCR) relation has to be developed for each soil type on the site. It is also dependent on being able to determine the OCR of the soil accurately, and this is often difficult because the pre-consolidation pressure of the soil as determined in oedometer tests is affected by sample disturbance. The recompression technique on the other hand is relatively simple and is recommended. It is emphasised that a lesser number of samples tested by recompression or SHANSEP techniques is far preferable to a large number of cheaper UC or UU tests.

The recompression and SHANSEP methods require a knowledge of the at rest earth pressure coefficient, Ko, *in situ*. This can be determined by:

- Self boring pressuremeter (SBPM) tests
- Use of published relationships between Ko and effective friction angle ϕ', plasticity index I_p, and over-consolidation ratio OCR.

The SBPM method is preferred as it gives a direct measure of Ko, but is relatively expensive and in many cases use of published relationships is sufficient. In any case SBPM results should be checked against the other methods such as:

(a) *For normally consolidated clays*
Jaky (1944) derived the theoretical relationship

$$Ko_{NC} = \left(1 + \frac{2}{3}\sin\phi'\right)\left(\frac{1 - \sin\phi'}{1 + \sin\phi'}\right) \tag{6.10}$$

where ϕ' = effective friction angle.

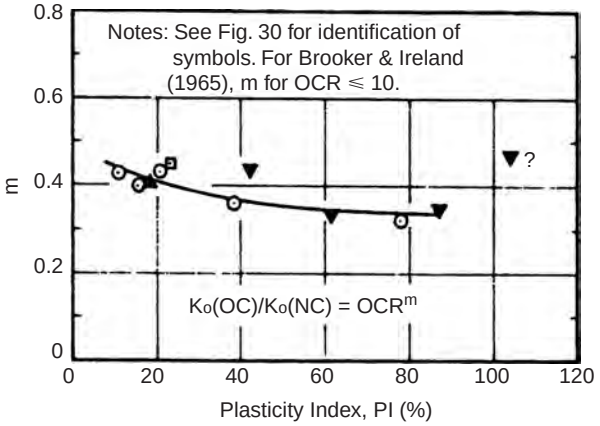


Figure 6.21 Coefficient m relating Ko and OCR vs plasticity index (Ladd *et al.*, 1977).

An approximation which is more commonly used is:

$$Ko_{NC} = 1 - \sin\phi' \tag{6.11}$$

Ladd *et al.* (1977) showed that this latter relationship was accurate within ±0.05.

(b) *For over consolidated clays*
Measure the pre-consolidation pressure (σ'_{vp}) in oedometer tests and calculate the OCR ($= \sigma'_{vp}/\sigma'_{vo}$). Then calculate Ko_{OC} from Equation 6.12 (Ladd *et al.* (1977))

$$Ko_{OC}/Ko_{NC} = OCR^{m} \tag{6.12}$$

where Ko_{NC} is estimated from the Jaky formula, m is determined from the Figure 6.21.

Wroth (1984) shows that Ko varies in loading and unloading emphasising that the Ko_{OC}/Ko_{NC} relationship is only applicable for reloading. On this basis the graphs would not be applicable to soils subject to over-consolidation by desiccation. For these soils, Ko should be determined by SBPM tests for important projects.

The undrained shear strength is dependent on the rate of shearing. Figure 6.22 shows the general effect, with the strength measured increasing with the strain rate.

Germaine and Ladd (1988) indicate that most laboratories (in their experience) use a strain rate of 0.5% to 1.0% of the sample height per hour, for soft cohesive soils and that this is reasonable based on case histories of undrained failures.

Note this is a lot slower than typically used in many laboratories. A strain rate of 1% per minute, often used for undrained testing, will over-estimate the undrained strength by about 20 ± 10% for low OCR clays and up to 50% for high OCR clays. In view of this, it is recommended that a strain rate of 1% per hour be specified.

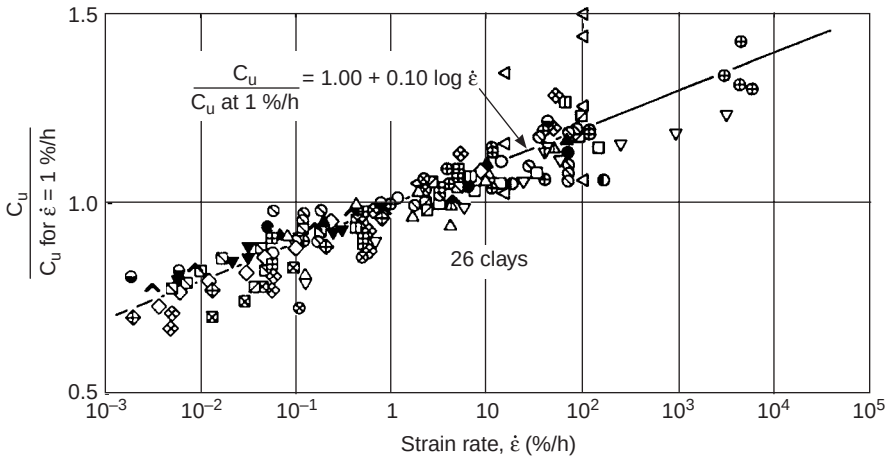


Figure 6.22 Influence of strain rate on the undrained shear strength (Cu ≡ Su) measured in triaxial compression (Kulhawy and Mayne, 1990).

6.1.6 Estimation of the undrained strength from the Over-Consolidation Ratio (OCR), at rest earth pressure coefficient Ko, and effective stress strengths

6.1.6.1 *Estimation of undrained strength from OCR*

Jamiolkowski *et al.* (1985) using data from Koutsoftas and Ladd (1984) showed that the normalized shear strength Su/σ'_{vc} versus OCR graphs for some soils were close to linear on a log-log plot. Hence:

$$Su/\sigma'_{vc} = S(OCR)^m \tag{6.13}$$

where σ'_{vc} = effective vertical stress in the test; $S = Su/\sigma'_{vc}$ for normally consolidated clay

$$\text{or } \frac{(Su/\sigma'_{vc})_{OC}}{(Su/\sigma'_{vc})_{NC}} = OCR^m \tag{6.14}$$

Jamiolkowski *et al.* (1985) show that for non-cemented clays with a plasticity index I_P less than 60%, $m \approx 0.8$ and $(Su/\sigma'_{vc})_{NC} = 0.23 \pm 0.04$. So:

$$(Su/\sigma'_{vc})_{OC} = (0.23 \pm 0.4)OCR^{0.8} \tag{6.15}$$

This method can be used as an initial estimate of strength, or as a check on other methods.

6.1.6.2 *Estimation of undrained strength from effective stress shear parameters*

Wroth and Houlsby (1985) give for normally consolidated soils:

$$(Su/\sigma'_{vo})_{NC} = 0.5743 \frac{3 \sin \phi'_{tc}}{3 - \sin \phi'_{tc}} \tag{6.16}$$

where subscript tc indicates triaxial compression testswith isotropic consolidation.

Davis in Thorne (1984) gave the relationship:

$$Su_{tc} = \frac{c' \cos \phi' - U_o \sin \phi'}{1 - (1 - 2A_f) \sin \phi'} \tag{6.17}$$

where

$$U_o = \frac{-(1 + 2Ko)\,\sigma'_{vo}}{3} = \frac{-(\sigma_{vo'} + 2\sigma'_{Ho})}{3};$$

A_f = Skempton 'A' at failure, c' = effective cohesion, ϕ' = effective friction angle, Ko = earth pressure coefficient at rest.

This relationship is applicable to both normally consolidated and over-consolidated clay, with A_f and Ko dependent on the degree of over consolidation.

6.1.7 Estimation of the undrained strength of cohesive soils from *in situ* tests

The undrained strength of cohesive soils in a dam foundation or within the embankment can be determined by *in situ* test methods. These include:

- Cone Penetration Test (CPT) and Piezocone Test (CPTU),
- Vane Shear,
- Self boring Pressuremeter.

6.1.7.1 *Cone Penetration and Piezocone Tests*

The undrained strength of clays can be estimated from the cone resistance q_c of the static cone penetrometer test (CPT).

The undrained strength Su is related to q_c by

$$Su = \frac{q_c - \sigma_{vo}}{N_K} \tag{6.18}$$

where q_c = cone resistance
σ_{vo} = *in situ* total overburden stress
N_K = empirical cone factor.

Lunne and Kleven (1981) presented graphs showing that for normally consolidated clays, N_K varies depending on the plasticity index of the soil. When uncorrected vane shear strengths are used as the base for correlation the values shown in Figure 6.23 are obtained. Note that in Figure 6.23 Po = *in situ* total vertical stress (= σ_{vo}).

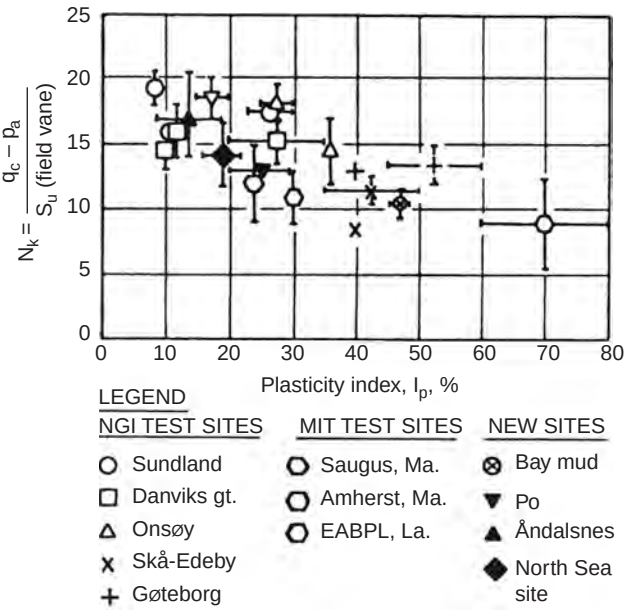


Figure 6.23 Empirical cone correlation factor N_K-based on uncorrected vane shear strengths – Lunne and Kleven (1981).

Aas *et al.* (1986) showed that when the vane shear strengths were corrected using Bjerrum's (1973) correction factor μ, where $Su = \mu Suv$, the cone correction factor N_K values were less dependent on the plasticity index of the soil, as shown in Figure 6.24.

Aas *et al.* (1986) point out that the N_K values proposed by Lunne and Kleven (1981) and others, are not corrected for the effect of pore pressure, and since they use different cones with varying geometry, and the pore pressure generated depends on clay type, much of the scatter in N_K values can be explained by these effects.

As summarised in Jamiolokowski *et al.* (1985) and in Aas *et al.* (1986) water pressure acting on the cone modifies the cone resistance by an amount which depends on the cone geometry and the pore pressure generated as the cone is inserted. The corrected cone resistance q_r is calculated from:

$$q_T = q_c + u(1 - a) \tag{6.19}$$

where q_T = corrected cone resistance
q_c = measured cone resistance
u = pore pressure at cone
a = area ratio = A_N/A_T (see Figure 6.25)

$$Su = \frac{q_T - \sigma_{vo}}{N_{KT}} \tag{6.20}$$

where N_{KT} is the empirical cone factor related to q_T.

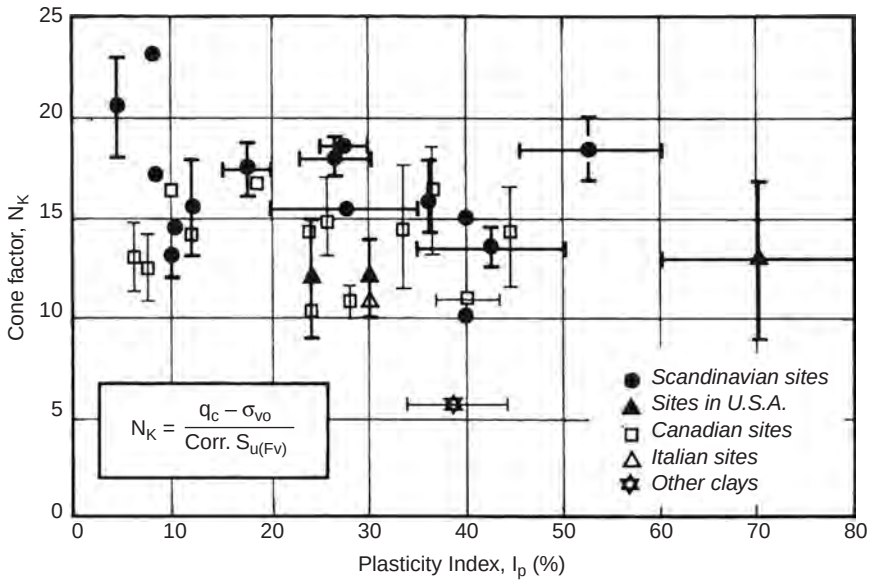


Figure 6.24 Empirical cone correction factor N_K based on corrected vane shear strengths – Aas *et al.* (1986).

Aas *et al.* (1986) present cone factors N_{KT} for several sites in Norway. These are reproduced in Figure 6.26.

It can be seen that N_{KT} varies from about 12 to 20 when correlated with the laboratory undrained strength (the average of TC, TE and DSS tests), and 12–22 when correlated to corrected field vane strength; the latter larger variation due probably in part to scatter in the vane shear strengths.

N_K and N_{KT} values show a much larger range for overconsolidated clays (e.g. Figure 6.26). N_K values of 15–20 are commonly adopted for oversonsolidated clays.

It can be seen from the above discussion that the static cone penetrometer or piezocone can be used to estimate undrained strength, but the value of the correlation factor N_K or N_{KT} is uncertain, and strengths cannot be estimated with a high degree of accuracy. The potential error is higher in overconsolidated clays.

In practice the static cone should only be used for assessment of undrained strength in slope stability problems in clay when it has been calibrated on the site by comparison with good quality laboratory undrained strengths.

The published correlations are too variable for the cone to be used alone for undrained strength assessment in slope stability other than for preliminary design with an appropriate factor of safety. In foundation design, where factors of safety adopted are usually greater than in slope stability analysis, cones may be used alone, but conservative NK factors should be adopted.

Piezocone equipment has become more readily available in recent years and should be used where practicable because the extra information obtained from the dynamic pore pressures as the cone is pushed into the soil can be a valuable guide as to whether

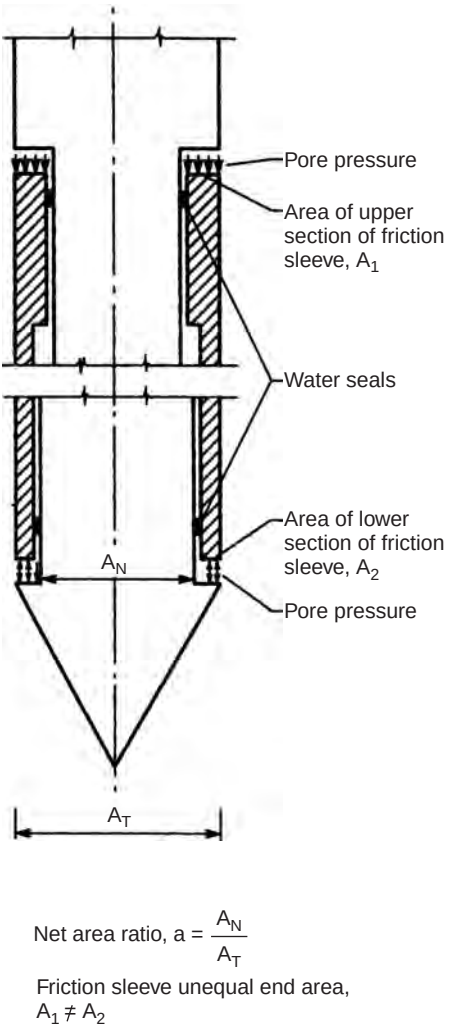

Pore pressure
Area of upper section of friction sleeve, A_1
Water seals
Area of lower section of friction sleeve, A_2
Pore pressure
A_N
A_T
Net area ratio, $a = \frac{A_N}{A_T}$
Friction sleeve unequal end area, $A_1 \neq A_2$

Figure 6.25 Vertical components of pressures acting on a cone penetrometer – Campanella and Robertson (1988).

the soils are contractive or dilative. This is particularly important in non-plastic soils where liquefaction may be an issue.

Compacted earthfill in dams will be over-consolidated, so conservatism should be exercised in selecting the N_K or N_{KT} values. It should also be noted that the correlations between undrained strength and cone resistance are developed for saturated soil conditions. When the CPT or CPTU is used in dams, the soils may be partially saturated above the phreatic surface. In view of these problems, where the undrained strength is needed with some accuracy, the CPT or CPTU should be calibrated against laboratory tests on samples from the dam.

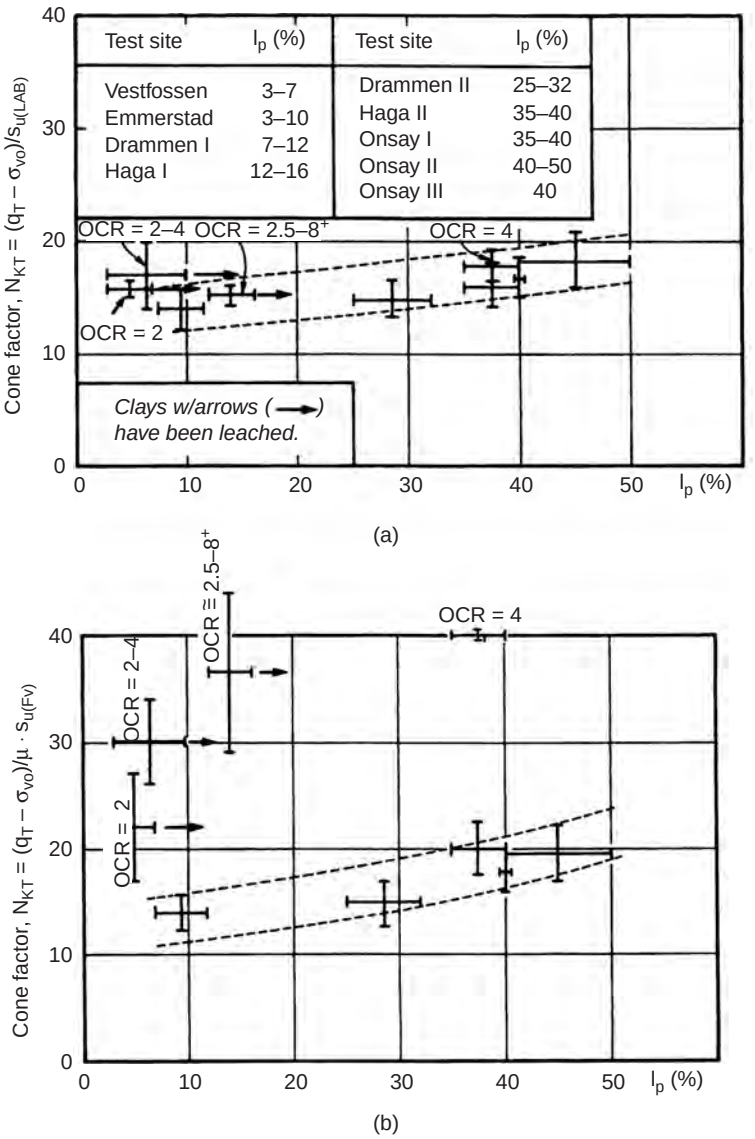


Figure 6.26 Cone factor N_{KT} for normally consolidated and overconsolidated clays – Aas *et al.* (1986) (a) Cone factor vs laboratory undrained strength (Piezocone), (b) Cone factor vs corrected vane shear strength (Piezocone).

The CPT and CPTU have the advantage of providing a continuous profile of q_c and hence a continuous profile of undrained strength. The cone resistance is however affected by the soil up to 10 to 20 diameters ahead of the cone, and may therefore under or overestimate the q_c (or q_T) depending on the situation.

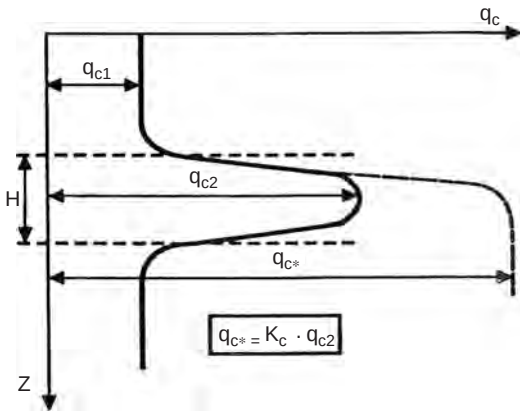


Figure 6.27 Suggested correction to CPT cone resistance in thin sand layers (Lunne *et al.*, 1997, based on Vreugdenhil *et al.*, 1994).

Based on a simplified elastic solution, Vreugdenhil *et al.* (1994) provided a method to correct cone data for thin layers. They showed that the error in the measured cone resistance within thin stiff layers is a function of the thickness of the layer as well as the stiffness of the layer relative to that of the surrounding soil. The relative stiffness of the layers is reflected by the change in cone resistance from the soft surrounding soil (q_{c1}) to that in the stiff soil layer (q_{c2}). Based on this work, a corrected cone resistance (q_{c*}) is derived as:

$$q_{c*} = K_c \cdot q_{c2} \tag{6.21}$$

where: K_c = a correction factor for the cone resistance as a function of the layer thickness.

The corrections recommended by Vreugdenhil *et al.* (1994) were considered by Lunne *et al.* (1997) to be rather large, so they recommended a more conservative correction which is shown in Figure 6.27 and given by the following expression:

$$K_c = 0.5\left(\frac{H}{1000} - 1.45\right)^2 + 1.0 \tag{6.22}$$

where: H = layer thickness, in mm, q_{c2} = cone resistance in the layer, q_{c1} = cone resistance in the soil surrounding the layer.

It should be noted that these expressions only apply for correction for resistance of a stiff layer within soft layers. The correction for a soft layer within a stiff soil is not significant.

The CPT or CPTU may not be able to penetrate well compacted earthfill. They will refuse on gravelly soils.

CPT or CPTU probes through dam cores should be backfilled with cement – bentonite grout inserted by pushing the rods without the cone on them or a steel

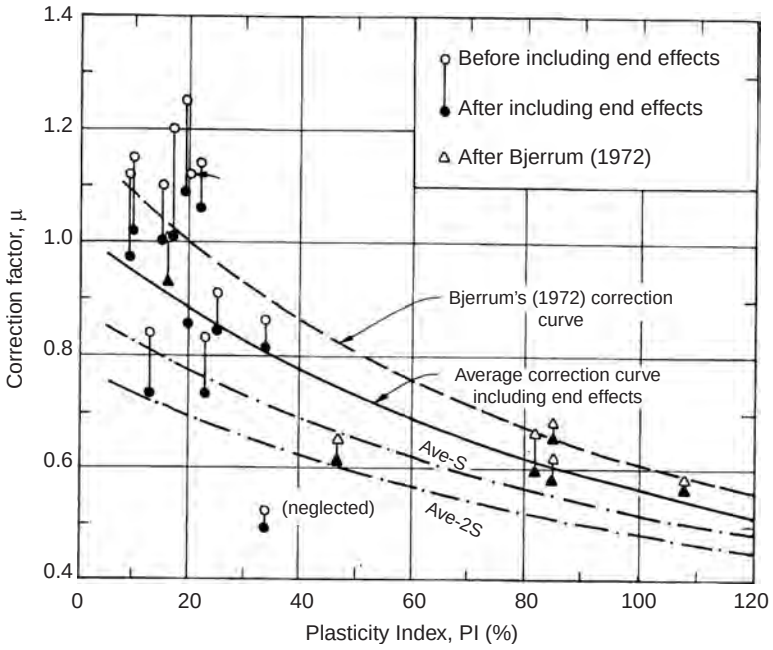


Figure 6.28 Vane shear correction factors (Azzouz *et al.*, 1983).

pipe down the hole, and injecting the grout at low pressures while withdrawing. Care should be taken to avoid hydraulic fracture and in some situations it may be judged too risky to probe through the core.

6.1.7.2 *Vane shear*

The vane shear may be used to measure the undrained strength of very soft to firm (Su less than about 60 kPa) cohesive soils. The equipment and procedures are described in many texts (e.g. Clayton *et al.*, 1995; Walker, 1983).

It is normal practice to correct the strength measured by the vane to account for anisotropy and rate of shearing. Bjerrum (1973) developed a method for doing this using back-analysis of case studies of embankment failures. Azzouz *et al.* (1983) reviewed the same failures as Bjerrum and some additional failures. They also considered 3-Dimensional (3D) effects. Azzouz *et al.* (1983) concluded that the correction factors obtained when 3-D effects were included gave better estimates of the actual strength as measured by the SHANSEP technique, and recommended use of the "average correction curve including end effects" for conventional 2-Dimensional analysis shown in Figure 6.28.

On balance it seems wise to apply the Azzouz *et al.* (1983) correction using the equation $Su = \mu S_{uv}$ where Su = design undrained shear strength; S_{uv} = undrained strength measured by the vane; μ = correction factor from Figure 6.28, but to also

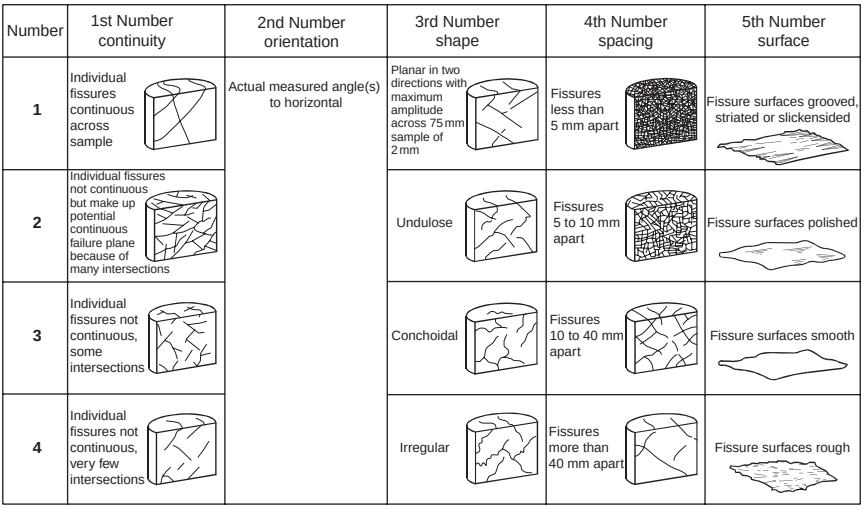

Number
1st Number continuity
2nd Number orientation
3rd Number shape
4th Number spacing
5th Number surface
1
Individual fissures continuous across sample
Actual measured angle(s) to horizontal
Planar in two directions with maximum amplitude across 75 mm sample of 2 mm
Fissures less than 5 mm apart
Fissure surfaces grooved, striated or slickensided
2
Individual fissures not continuous but make up potential continuous failure plane because of many intersections
Undulose
Fissures 5 to 10 mm apart
Fissure surfaces polished
3
Individual fissures not continuous, some intersections
Conchoidal
Fissures 10 to 40 mm apart
Fissure surfaces smooth
4
Individual fissures not continuous, very few intersections
Irregular
Fissures more than 40 mm apart
Fissure surfaces rough

Figure 6.29 Fissure Classification Explanation Sheet (Walker *et al.*, 1987, after Coffey Geosciences).

calibrate the vane with good quality laboratory testing on samples from the site. For design of embankments on soft clays it is recommended the Ferkh and Fell (1994) method be adopted, but with more conservative factors of safety than recommended in the paper as detailed in Section 11.3.2.

6.1.7.3 *Self Boring Pressuremeter*

The undrained strength of cohesive soils can be measured using the Self Boring Pressuremeter (SBPM).

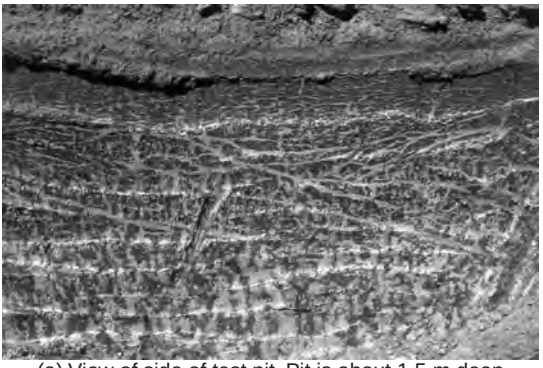
However the measured strength is affected by disturbance, being larger the greater the disturbance and the extent of disturbance is unpredictable. In view of this use of the SBPM to estimate Su is not recommended.

6.1.8 Shear strength of fissured soils

6.1.8.1 *The nature of fissuring, and how to assess the shear strength*

Fissures are discontinuities in soils which have similar implications to the strength and permeability that joints have in rock masses – they reduce the soil mass strength, and (usually) increase the soil mass permeability. Fissured soils are sometimes present in the foundations of dams – either left in place by design, or because they were not removed by those who built the dam, not realising the effect on the shear strength.

Fissures in soil have a wide range of characteristics relating to the properties of the soil mass, the depositional and weathering history and climate (when the fissures were formed and now). It is useful to log, and classify the fissures systematically.



(a) View of side of test pit. Pit is about 1.5 m deep



(b) View of pieces of the fissured soil.
The large piece is about 300 mm across.

Figure 6.30 Weathered Tertiary clay, which exhibits shrink-swell (desiccation) induced fissures, the major sets being inclined at about 20° to the horizontal, consistent with passive failure on swelling. (a) view of side of test pit; (b) view of pieces of the fissured soil.

Walker *et al.* (1987) provide a fissure classification system which is simple and practical to use. The main components are shown in Figure 6.29. Figures 6.30 and Figure 6.31 show examples of fissured soils.

Fissures are caused by one or more of the following:

(a) Stress relief due to erosion or retreat of glaciation.
(b) Freeze-thaw.
(c) Shrink-swell (desiccation) due to seasonal moisture content changes.
(d) Differential consolidation and settlement in sedimentary soils.

It is important to recognise that the fissures are often formed in a different geological and climatic condition from the present – e.g. in periods of glacial retreat, or when the sea level was lower than now. The fissured soils may in some cases be overlain by more recent sediments which are not fissured (e.g. Thorne, 1984).



Figure 6.31 Fissures in pieces of weathered volcanic tuff. The fissures are induced by desiccation when the ground surface was exposed above sea level. Largest pieces are about 450 mm across.

The shear strength of fissured soils is dependent on:

- The peak, fully softened and residual strength of the soil substance.
- The continuity, orientation, shape and spacing of the fissures and the nature of the fissure surfaces.
- These are in turn related to the origin of the fissures.

For example, smooth, planar fissures may only reduce the soil mass strength to the fully softened strength, but polished and slickensided, planar or undulose, continuous fissures, will reduce the strength in the direction of the fissures to near the residual strength.

It has been common practice to adopt the fully softened strength to approximate the mass strength of the soil. This is based on back analyses of slides in English clays (Skempton, 1977; Chandler, 1984). The authors' experience is that in some soils, e.g. fissured residual basalt and tuff (MacGregor *et al.*, 1990; Moon, 1992) and fissured marine clays (Thorne, 1984), there was evidence from back analysis of failures, or from laboratory testing and mapping of the fissures, that the mass strength could be considerably lower than the fully softened. Table 6.2 summarises the data. Raby Bay and Plantes Hill both had failures which could be back-analysed (albeit with rather poor information on pore pressures at the time of failure).

Stark and Eid (1997) present data from 14 cases which show back-analysed mobilised shear strengths, for fissured clays as being about the average of the fully softened and residual strengths for soils with a liquid limit between 50–130%. They explain this in terms of progressive failure, rather than the nature of the fissures controlling the shear strength, which is considered more likely by the authors.

It is recommended that the following procedure be followed to assess the shear strength of fissured clays.

(a) Map the fissures for orientation, continuity and surface characteristics as detailed in Figure 6.29.

Table 6.2 Shear strengths of fissured soils from three projects in Australia.

		Shear strengths					
		Peak		*Residual*		*Adopted*	
Project	*References*	c'	ϕ'	c'_R	ϕ'_R	c'	ϕ'
Raby Bay(1)	Moon (1992)	5	24	2	8	0	13
Plantes Hill(2)	MacGregor *et al.* (1990)	10	25	0	11	5	18
Botany Bay(3)	Thorne (1984)	25	24	0	13	5	15

(1) Residual soils derived from tuff. Strengths are those used while the first author was involved in project. One failure was back analysed to give $c' = 0$ kPa, $\phi' = 13°$.
(2) Residual soil derived from basalt.
(3) Marine clay fissured by desiccation and differential consolidation.

(b) Analyse the orientation using stereo projection methods as one does for rock joints. MacGregor *et al.* (1990) give an example.
(c) Carry out laboratory tests to determine the peak, fully softened and residual strengths and, if possible, the strength along fissures (this procedure is described below).
(d) Assess the strength of the soil mass parallel to the fissure surfaces (ϕ'_f) accounting for the continuity and nature of the fissure surfaces, e.g. polished slickensided fissures would be assigned residual strength, and if they were 50% continuous, the strength determined as approximating to the average of residual and fully softened strength. Normal to the fissures, and for rough fissure surfaces, fully softened strength would apply.

This procedure is highly judgemental, and should where practicable, be backed up with back analyses of failures in the same geological materials.

For failure surfaces which are not parallel to the fissure direction but inclined at an angle α to the fissure orientation as shown in Figure 6.32 the shear strength along the failure surface C-D can be taken as $\phi'_f + \alpha$ in a similar manner to how the shear strength on rock defects is estimated as the basic friction angle plus angle of dilation. The limiting strength will be the fully softened strength of the soil because the soil strength will take over for steeply angled fissures.

For failure surfaces B-C and D-E the fissure strength ϕ'_f applies.

As most fissures formed by desiccation are oriented at relatively steep angles to the horizontal if detailed investigation can demonstrate there are no low angle fissures present, the fissures will have little effect on failure surfaces which are predominantly horizontal. This approach was used on Ross River Dam foundation.

The undrained strength is also affected by the presence of fissures. This is best assessed by using the procedures described in Section 6.1.6 to estimate the strength from the effective stress strengths (which have been assessed allowing for the fissures).

CPT, CPTU and Vane Shear Tests do not allow the soil to fail along the fissures, so they will generally measure the soil substance strength, not the fissured mass strength, and hence will over-estimate the strength.

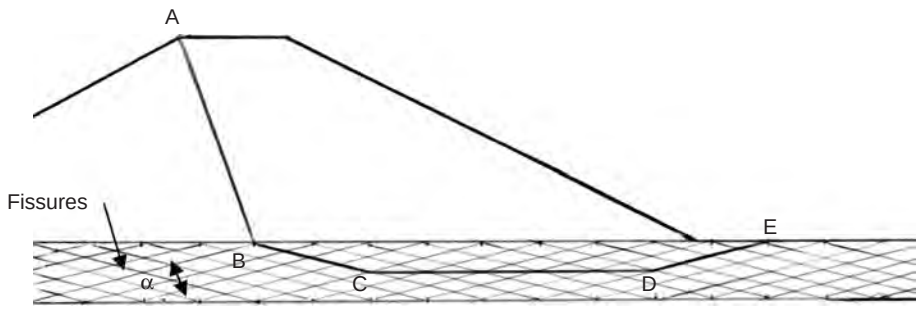


Figure 6.32 Method for estimating the shear strength of fissures soils based on fissure orientation.

6.1.8.2 *Triaxial testing of fissured soils*

The shear strength of a fissured soil mass can be determined by triaxial testing, provided the sample is sufficiently large to contain a representative selection of the fissures, and the fissures are oriented in such a way that failure will occur along the fissures, rather than the soil substance.

In practice this means (Rowe 1972) that large diameter samples – at least 100 mm, and preferably 250 mm diameter are needed. This is usually impractical because even if the samples can be obtained by block sampling, the test duration would be very long.

Hence it is usually more practicable to determine the fissure strength, either by judgement, from the nature of the fissures, and the peak, fully softened and residual strength, as described above, or by triaxial testing, and then use the orientation and continuity of the fissures to assess the mass strength. The strength of the fissure surfaces can be determined by the procedure described by Thorne (1984). This can be applied to block samples or thin walled tube samples and consists of:

- Test 75 mm diameter samples, being the maximum size which can readily be obtained with thin wall samples tubes;
- Carefully examine the samples before and after testing to determine whether shearing has occurred on a fissure surface or through the soil substance, and note the orientation of the failure surface;
- Determine the shear stress on the fissure surface from the Mohr's circle using the construction shown on Figure 6.33;
- Plot the strengths *on the fissure surfaces* in this manner to determine the design fissure strength (as a p′-q plot of points).

Thorne (1984) shows that whether a sample fails on a fissure depends on the orientation of the fissure and the magnitude of the stresses. Figure 6.34 illustrates this.

- For Figure 6.34(a) 60° fissure: a, b and c all fail on fissure.
- For Figure 6.34(b) 25° fissure: Circle a fails through substance.

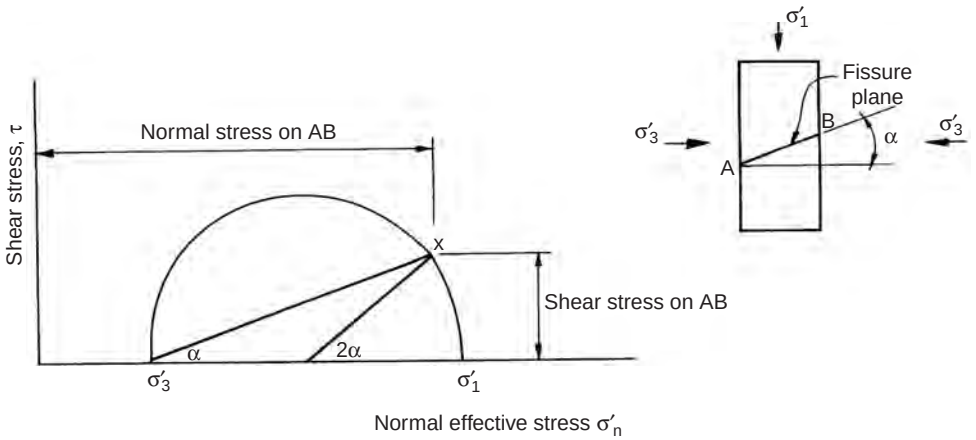


Figure 6.33 Resolution of stresses on failure surface.

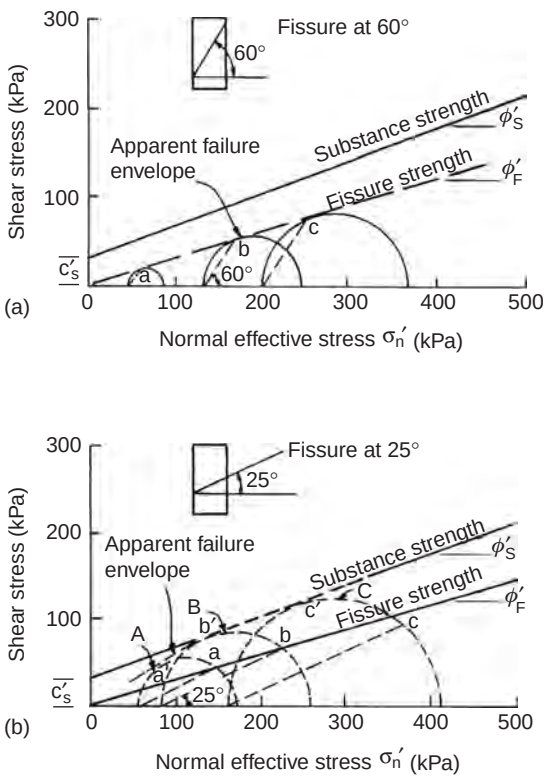


Figure 6.34 Mohr's circle envelopes for fissures at 60° and 25° (Thorne, 1984).

For fissured soils which can be sampled by block sampling from test pits, shafts or other excavations, it is often practical to test the actual fissure surface in a direct shear test to obtain the fissure strength.

On a practical note, it is very unusual to be able to see the fissures in soil samples prior to testing, so it is usually necessary to test a large number of samples to obtain sufficient samples which fail on fissure surfaces not intersecting the end plates. Some fissured samples tend to fall to pieces on ejection from a sample tube. This can be overcome to a degree by cutting down the sides of the sample tube, so the sample can be lifted out, rather than forced out of the end in the normal manner.

6.1.9 Estimation of the effective friction angle of granular soils

6.1.9.1 *Methods usually adopted*

The shear strength (ϕ') of silts, sands, and gravels in the dam foundation or within the embankment can be assessed in three ways:

(a) *In situ* testing, such as Standard Penetration Test (SPT), Cone Penetration Test (CPT) or Piezocone (CPTU).
(b) Sampling and laboratory testing (direct shear or triaxial).
(c) Estimation from published data, based on the particle size distribution and compaction layer thickness, number of passes of a roller and/or relative density (density index) of compaction.

Method (a) is commonly used for assessing the strength of foundation soils, and occasionally embankment materials e.g. in hydraulic fill, or upstream construction tailings dams.

Method (b) is seldom used in view of the fact that the peak effective friction angle (ϕ') is dependent on the relative density, so laboratory testing requires a knowledge of the in-place relative density of the soil which, for dam foundations, can only be found by *in situ* testing (SPT, CPT, CPTU). Also, it is very difficult to obtain undisturbed samples of granular soils. The normal practice therefore is to estimate the shear strength of those soils directly from the in-situ test, rather than doing laboratory tests. For very large embankment dams, it was common to do laboratory shear strength tests on the filters, but in reality this was not necessary, as they seldom influence stability very much. For outer zones ("shells") of embankments constructed of silt-sand-gravels, it would be good practice to test in the laboratory, at the relative density to be specified for construction.

Method (c) is commonly used for estimating the strength of filter zones and is sufficiently accurate in most cases.

Each of these is described in the following sections.

6.1.9.2 *In situ tests*

Standard Penetration Test (SPT)

The effective friction angle, ϕ' can be estimated directly from the SPT 'N' values (e.g. using the methods of Mitchell and Katti, 1981; Peck *et al.*, 1974) or by using the SPT to estimate the relative density and then estimating the strength from the relative density

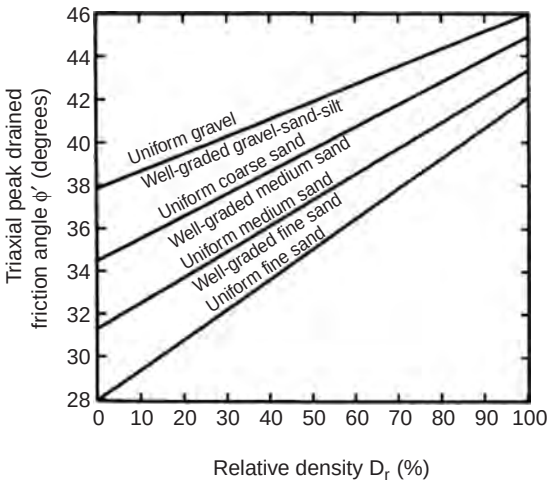


Figure 6.35 Peak effective friction angle of sands as a function of relative density and grain size (Schmertmann, 1978).

and general grain size characteristics. Schmertmann (1978) developed such a method which is shown in Figure 6.35. Lunne *et al.* (1997) indicate the laboratory tests on which Figure 6.35 is based were at a confining stress of 150 kPa, and that lower values of ϕ' should be used at high confining stresses.

The relative density is best estimated using the method suggested by Skempton (1986). He reviewed the available information and proposed a method where the SPT value is corrected for the energy of the hammer, and to an effective overburden pressure of 100 kPa by:

$$N_{60} = Nm\, ERm \tag{6.23}$$

where Nm = measured SPT 'N' value
N_{60} = 'N' value corrected to a hammer of 60% efficiency
ERm = Energy Rating of the SPT hammer
and

$$(N_1)_{60} = C_N N_{60} \tag{6.24}$$

where C_N = factor to correct N_{60} to 100 kPa effective overburden stress

$$= \frac{200}{100 + \sigma'_{vo}} \quad \text{for fine, normally consolidated sand} \tag{6.25}$$

$$= \frac{300}{200 + \sigma'_{vo}} \quad \text{for coarse, normally consolidated sand} \tag{6.26}$$

Table 6.3 Skempton (1986) method for estimating relative density (D_R) from $(N_1)_{60}$ value.

Relative density (%)	*Condition*	$(N_1)_{60}$
0	Very loose	0
15	Loose	3
35	Medium dense	8
65	Dense	25
85	Very dense	42
100	Very loose	58

$$= \frac{170}{70 + \sigma'_{vo}} \quad \text{for overconsolidated sand} \tag{6.27}$$

where σ'_{vo} = effective overburden stress at the level of the SPT test (in kN/m^2).

Skempton (1986) then uses the $(N_1)_{60}$ value to estimate relative density from Table 6.3.

For $D_R \geq 35\%$, this can be approximated by

$$(N_1)_{60}/D_R^2 = 60 \tag{6.28}$$

This method applies to naturally occurring sands, other than sands of very recent deposition.

The method requires measurement of the energy rating of the SPT equipment. This should be done on the site being investigated as the energy transmitted to the rods and the SPT varies depending on the type of equipment, and the condition of drill rods etc. This is particularly important where SPT are being used to assess liquefaction.

Skempton (1986) also considers the effect of the age of the deposit on the SPT values. Discussing this, Jamiolkowski *et al.* (1988) produced Figure 6.36 which indicates that $(N_1)_{60}/D_R^2$ is dependent on the age of the deposit and will be less for recently deposited soil than for the more common aged soil.

This also means that for recently dredged fills, or deposits of mine tailings, a $(N_1)_{60}$ value of say 10, will imply a relative density of ≈50%, compared to 40% for an aged natural deposit.

Cone Penetration (CPT) and Piezocone (CPTU) Tests

The effective friction angle ϕ' can be determined directly from the CPT and CPTU using charts which have been developed by several authors e.g. Robertson and Campanella (1983a, b), Marchetti (1985, Kulhawy and Mayne (1988), Jamiolkowski *et al.* (1985), and Olsen and Fan (1986). Lunne *et al.* (1997) reviewed the methods and concluded that the Robertson and Campanella (1984) and two other methods were to be preferred. The Robertson and Campanella (1984) method is simple to use and is shown in Figure 6.37.

They indicate that:

– The method applies to normally consolidated, uncemented, moderately incompressible (grains), predominantly quartz sands but can be used for silty sand, sandy silt, sand and gravelly sand.

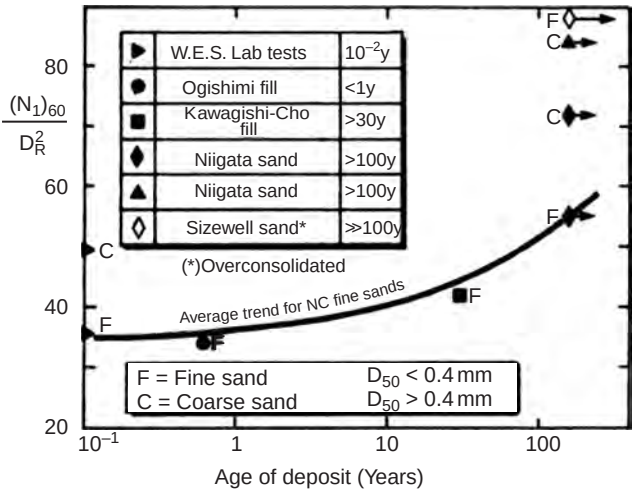


Figure 6.36 Effect of aging of the soil on SPT resistance (Jamiolkowski *et al.*, 1988).

- For highly over-consolidated sands ϕ' may be up to 2° lower than predicted from Figure 6.37.
- For highly compressible sand (e.g. sand with many shells, or mica pieces) the method predicts conservatively low friction angles. For the sands included in Robertson and Campanella's analysis the effect could be up to 3°.

The *in situ* methods described give peak strength (not large strain, steady state strength) and do not appear to account for any curvature of the strength envelope. Hence if it is intended to construct a large dam on such a foundation, it may be necessary to account for this by using somewhat lower strengths. The Jamiolkowski *et al.* (1985) method allows for this.

6.1.9.3 *Laboratory tests*

Laboratory tests to determine the effective friction angle of granular materials are described in ASTM and British standards and in Head (1985).

The most difficult aspect is to be able to prepare the sample at the correct relative density. Coarse grained materials (containing gravel) may have to be re-graded so they can be tested. The normal procedure is to prepare soils with a particle size distribution finer than and parallel to the actual.

6.1.9.4 *Empirical estimation*

A good guide to the strength of compacted sand and sandy gravel is to use the relative density specified (or expected) and particle size and use Figure 6.35.

Normal compaction of granular filters in dams e.g. 4 passes of a 10 tonne smooth drum vibratory roller, in layers 500 mm compacted thickness, should give a relative

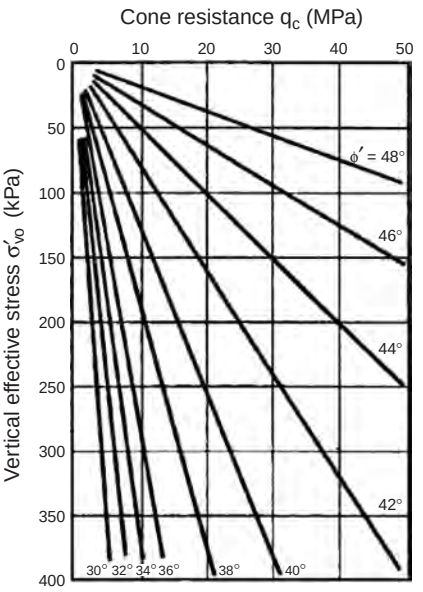


Figure 6.37 Robertson and Campanella's (1984) method for estimating ϕ' for cohesionless soils from the cone penetration test (CPT).

density of at least 70%, so the effective friction angle should be around 38° for fine sand and 40° for well graded filter sands and sandy gravels.

In reality at low confining stresses e.g. less than 200 kPa, ϕ' is likely to be 2° to 5° higher than these figures. If the figures become critical to the stability analysis, they should be checked by laboratory testing.

The friction angle of coarse grained granular soils (gravelly sands and gravels) can be estimated from Figure 6.38 from USBR (1998). Note gravel is based on 4.75 mm size.

6.1.10 Shear strength of partially saturated soils

See Section 11.2.4 for a discussion on this topic.

6.2 SHEAR STRENGTH OF ROCKFILL

The shear strength of rockfill is dependent on the effective normal stress, and to a lesser extent on the bulk dry density, void ratio or degree of compaction; the unconfined compressive strength of the rock, the uniformity coefficient of the grading, maximum grain size, fines content, and particle shape (Marsal, 1973, Douglas, 2003). Table 6.4 summarises the effect of these variables. Figure 6.39 shows the methods for representing the shear strength.

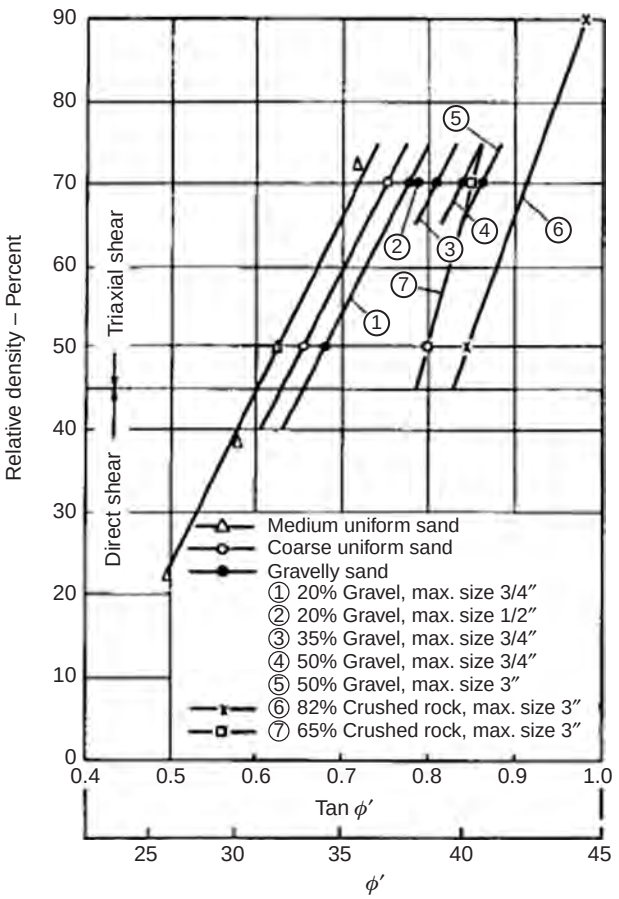

Relative density – Percent
Triaxial shear
Direct shear
Medium uniform sand
Coarse uniform sand
Gravelly sand
① 20% Gravel, max. size 3/4″
② 20% Gravel, max. size 1/2″
③ 35% Gravel, max. size 3/4″
④ 50% Gravel, max. size 3/4″
⑤ 50% Gravel, max. size 3″
⑥ 82% Crushed rock, max. size 3″
⑦ 65% Crushed rock, max. size 3″
Tan φ′
φ′

Figure 6.38 Peak friction angle for coarse grained soils as a function of relative density and grain size (USBR, 1998).

Douglas (2003) carried out an extensive analysis of the factors affecting the shear strength of rockfill. This involved establishing a large database from published and unpublished triaxial testing, and statistical analysis.

Figure 6.40 shows the variation of secant friction angle ϕ'_{sec} with normal stress σ_n for dense rockfill. The strong dependence of ϕ'_{sec} on σ_n is evident, as is the wide variation in ϕ'_{sec}.

Douglas (2003) carried out non-linear statistical analyses on the database. The estimation method used was least squares. An analysis of the form shown below was found to be the most effective for relating ϕ'_{sec} to σ_n.

$$\phi'_{sec} = a + b\sigma_n'^{c} \tag{6.29}$$

Table 6.4 Factors affecting the shear strength of rockfill.

Variable	*Description of effect on shear strength*	*Importance of variable*	*References*
Confining pressure	Mohr-Coulomb shear strength envelope is curved, with lower friction angles at high confining stress	Major – see Figures 6.39, 6.40	Douglas (2003), Indraratna *et al.* (1993, 1998), Charles and Watts (1980), Marsal (1973)
Density or void ratio	Shear strength increases with increased density, lower void ratio. The effect is greatest at low confining stress, with little effect at high confining stress	Medium to minor	Douglas (2003); Marsal (1973), Nakayama *et al.* (1982)
Unconfined compressive strength of the rock	Shear strength increases with increased rock strength below UCS 100 MPa but little effect for stronger rock. Little effect at low confining stress, medium effect at high confining stress	Minor to medium	Douglas (2003), Anagnosti and Popovic (1982)
Uniformity coefficient	Uniformly graded rockfill exhibits more curvature in the Mohr Coulomb strength envelope than well graded. No clear effect on strength (Douglas)	Minor	Sarac and Popovic (1985), Douglas (2003)
Maximum particle size	Most accept that shear strength increases with particle size (Chiu, Douglas), but some claim no effect (Charles *et al.*), or the opposite effect (Anagnosti and Popovic)	Minor	Chiu (1994), Charles and Watt (1980), Anagnosti and Popovic (1981)
Finer particles content	Shear strength reduced significantly with more than about 35–50% silty or clayey sand passing 2 mm size (USBR). Rockfill with >20% fines passing 0.075 mm had lower strength (Douglas)	Medium	USBR (1966), Douglas (2003)
Particle angularity	Angular particles have higher strength than sub angular and rounded particles at low confining pressures, little difference at high confining pressures	Minor	Douglas (2003), Sarac and Popovic (1985), Bertacchi and Berlotti (1970)

The analysis resulted in the following constants:

$$a = 36.43 - 0.267\text{ANG} - 0.172\text{FINES} + 0.756(c_c - 2) + 0.0459(\text{UCS} - 150) \tag{6.30}$$

$$b = 69.51 + 10.27\text{ANG} + 0.549\text{FINES} - 5.105(c_c - 2) - 0.408(\text{UCS} - 150) \quad -0.408 \tag{6.31}$$

$$c = -0.3974 \tag{6.32}$$

where ANG = (angularity rating −5) for angularity rating >5.5 otherwise 0, rounded particles have an angularity rating of 0, crushed angular rock a rating of 8. The rating is subjective (Douglas 2003).

FINES = percentage of fines passing 0.075 mm (%).

$$c_c = \text{coefficient of curvature} = \frac{d_{30}^2}{d_{10}d_{60}}$$

UCS = unconfined compressive strength of the rock substance (MPa).

This function resulted in a variance explained of 61.7%. The addition of the uniformity coefficient, c_u and the maximum and minimum particle sizes, d_{max} and d_{min}, did not result in a better fit to the data.

Douglas (2003) also carried out an analysis of the database using the principal stresses σ_1' and σ_3'. This showed an equation of the form shown in Equation 6.33 was found to be the most effective for relating σ_1' to σ_3'.

$$\sigma_1' = \text{RFI}\sigma_3'^{\alpha} \tag{6.33}$$

The analysis resulted in the following equations (based on 869 data sets and with a variance explained = 98.8%):

$$\alpha = 0.8726 \tag{6.34}$$

$$\begin{aligned}\text{RFI} = {} & 6.3491\text{RFI}_e + 0.48763\text{RFI}_{ANG} - 0.0027\text{RFI}_{d_{max}} \\ & -1.1568\text{RFI}_{FINES} + 0.30598\text{RFI}_{UCS}\end{aligned} \tag{6.35}$$

$$RFI_e = \frac{1}{1+e_i} \tag{6.36}$$

$$\text{RFI}_{ANG} = 1 \text{ if angular, otherwise} = 0 \tag{6.37}$$

$$\text{RFI}_{d_{max}} = d_{max}(\text{mm}) \tag{6.38}$$

$$\text{RFI}_{FINES} = \frac{e^{(\text{Fines}-20)}}{1+e^{(\text{Fines}-20)}} \text{ where fines is in \%} \tag{6.39}$$

$$\text{RFI}_{UCS} = \frac{e^{(\text{UCS}-110)}}{1+e^{(\text{UCS}-110)}} \text{ where UCS is in MPa} \tag{6.40}$$

For many analyses of stability it will be sufficient to use the data in Figure 6.34 and Equations 6.29 to 6.32 to select the strength of well compacted rockfills. A bi-linear strength envelope should be used to better represent the curved Mohr-Coulomb envelope. This can be plotted by calculating ϕ'_{sec} from Equations 6.29 to 6.32 at varying σ_n applicable to the stability analysis.

Loose dumped rockfill at low σ_n will have ϕ_{sec} about 4 degrees lower than calculated using this approach. At high σ_n the difference will be negligible. For more important projects, Equations 6.33 to 6.40 should be used, to give the relationship between σ_1' and σ_3'. To convert this to the more familiar c′, ϕ', use the method described in Section 16.3.2.5, Equations 16.16 to 16.18.

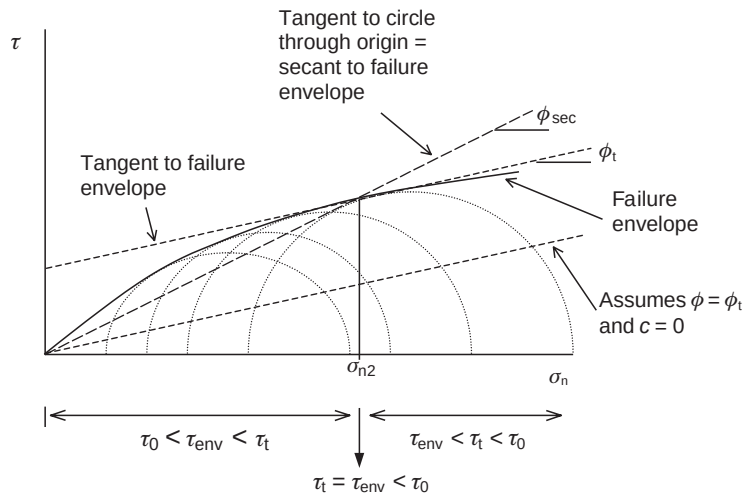


Figure 6.39 Methods for representing the shear strength of rockfill (Douglas, 2003).

If the rockfill has high percentages of finer particles (say greater than 25–30%, passing 2 mm), it would be wise to carry out triaxial tests, using scaled coarse fraction particle size distribution, but retaining the same percentage passing 2 mm.

For very large dams, it may be necessary to carry out triaxial tests on scaled gradings of the rockfill to confirm the strength, but only if the strength is critical (which in reality it seldom is).

6.3 COMPRESSIBILITY OF SOILS AND EMBANKMENT MATERIALS

6.3.1 General principles

6.3.1.1 *Within the foundation*

The compression of soils in the foundation of a dam, as load from the dam is applied during construction, has three components:

- Initial "elastic" compression.
- Primary consolidation, as the pore pressures induced by the weight of the dam is dissipated.
- Secondary consolidation, or creep, which takes place with no further change in pore pressures.

The theory and principles involved are covered in soil mechanics text books and will not be discussed here. There are some points to keep in mind:

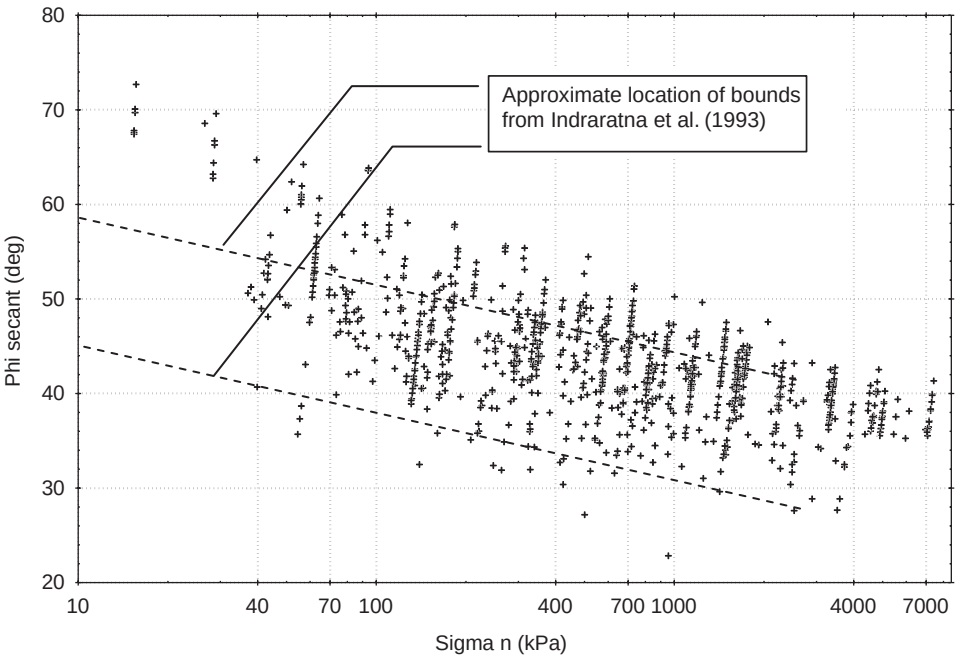


Figure 6.40 Variation of secant friction angle ϕ'_{sec} with normal stress σ_n (Douglas, 2003).

(a) The *in situ* vertical and horizontal stresses, σ'_{vo} and σ'_{HO} are affected by over-consolidation ratio ($OCR = \sigma'_{vp}/\sigma'_{vo}$, where σ'_{vp} = pre-consolidation pressure) of the foundation soils. These are influenced by periods of erosion of the site, lower groundwater levels than at present, desiccation from drying of soils exposed to the sun (or by freezing) and, in some areas, glaciation.

(b) The coefficient of consolidation, along with the drainage path distance determines the time for primary consolidation:

$$t = \frac{TH^2}{C_v} \tag{6.41}$$

where t = time for primary consolidation (years)
T = time factor
H = drainage path distance (metres)
C_v = coefficient of consolidation (m^2/year)

The coefficient of consolidation is dependent on the OCR (it is larger in the over-consolidated range of stresses, than in the normally consolidated range) and by fabric in the soil (e.g. fissures, fine layers of silt or sand interbedded with clay). Care must be taken to test sufficiently large samples to include the fabric e.g. by

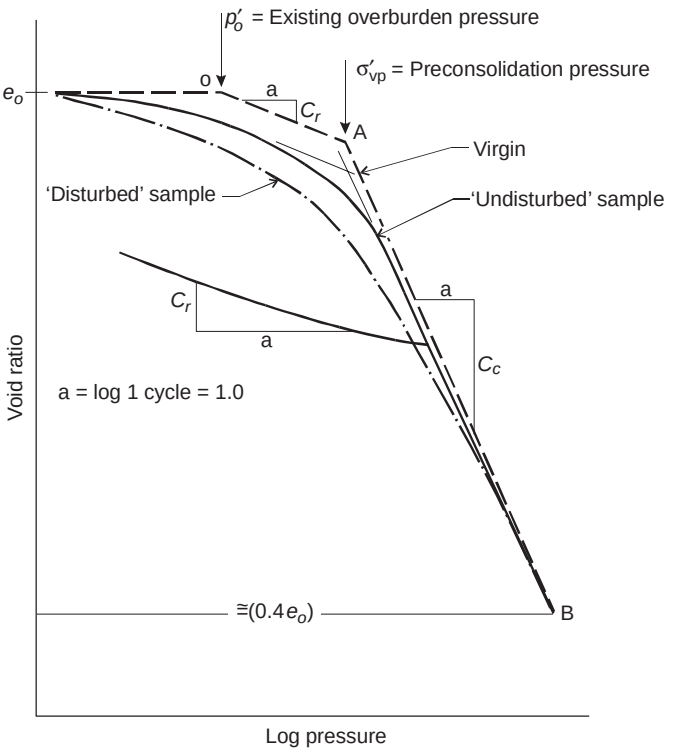


Figure 6.41 Void ratio versus log effective stress plot for a typical overconsolidated soil with the Schmertmann (1955) method shown to correct C_c for sample disturbance.

using triaxial consolidation. However, in these tests, the side drains may become a control on the rate of flow of pore water from the soil.

The drainage path H, is dependent on the thickness of the clay layer, and the presence of more permeable sandy, or silty layers within the clay. These can often best be located by cone penetration (CPT) or piezocone (CPTU) testing.

(c) Consolidation within the over-consolidated stress range (O-A in Figure 6.41) results in relatively small changes in void ratio (and hence consolidation) as the soil follows the unload-reload index C_r. Consolidation within the primary or normally consolidated range (A-B in Figure 6.41) gives larger changes in void ratio as the soil follows the compression index C_c. Hence, to estimate settlements, it is important to clearly define the pre-consolidation pressure σ'_{vp} in oedometer testing.

It is recommended that the Schmertmann (1955) correction to C_c be applied to allow for sample disturbance. This involves selecting point 0 from the existing effective over burden pressure σ'_{vo} from which the sample was taken and the *in situ* void ratio e_o, drawing OA parallel to the unloading curve with A on the pre-consolidation stress σ'_{vp}; and joining A to point B, which is on the consolidation plot at $0.4e_o$.

The pre-consolidation pressure measured in the laboratory should be checked against the history of the soil deposit, and indirectly checked using the relationship between the undrained strength of clays in the deposit, and the OCR (and hence σ_{vp}) using Equation 6.14.

6.3.1.2 *Within the embankment*

As for the foundation, the materials within the embankment have three components to their compression – initial "elastic", primary consolidation, and creep. Considering each type of material in the embankment:

Earthfill

Soils in modern embankment dams are compacted to a degree of saturation which is typically 90–95%, so their early behaviour is that of partially saturated soils. Much of the consolidation process consists of the gas in the voids being compressed, and forced into solution, rather than flow of water from the system. It therefore occurs relatively quickly and, as discussed in Section 6.2.2, most of the settlement in dams occurs during construction. It is incorrect to try to predict the behaviour of these zones by carrying out conventional oedometer consolidation tests (which use saturated soils), and classical Terzaghi consolidation theory, because these do not properly model the partially saturated behaviour, and impose zero lateral strain (Ko) conditions, which are often not correct, particularly if the earth core of the dam is narrow and the rockfill compressible giving little lateral constraint.

The compaction process typically over-consolidates the soil to at least 200 kPa, but this depends on the compaction equipment, soil and the compaction water content. Hunter (2003) and Hunter and Fell (2003d) determined from internal settlement data that for earthfill cores placed generally 0.5 to 1% dry of standard optimum moisture content, the pre-consolidation pressure was from 200 to 400 kPa for clayey earthfills, and 700 to 1,000 kPa for silty gravel, silty sand and clayey sand.

The theory and practice of partially saturated soil behavior is rapidly developing and readers should seek the latest information from the literature. Some useful references include Alonso *et al.* (1990), Bardin and Sides (1970), Khalili and Khabbaz (1998) and Loret and Khalili (2002).

Earthfill materials that are poorly compacted may undergo collapse compression upon first wetting. Guidance on estimating the amount of collapse compression is provided in Section 8.3.3.

Filters and sand-gravel zones

These will, if well compacted, be stiffer (have a higher modulus E) than earthfill and are usually stiffer than rockfill. The modulus will depend on the grading and degree of compaction. Because they are stiff, they may attract load by arching of the core. Some data on the modulus of compacted "gravel" fill in concrete face rockfill dams is given in Section 15.2.4 and this can be used to estimate the modulus of filters.

Rockfill

The rockfill compressibility is dependent on the grading, degree of compaction (layer thickness, roller weight and number of passes), rock substance strength and the effect of wetting on the substance strength. The modulus is best estimated from the performance of similar dams and is discussed in detail in Section 15.2.4.

The rockfill upstream of the core of the dam is subject to wetting by the reservoir and to cyclic loading as the reservoir levels fluctuate. Some rockfills are subject to "collapse settlement" due to softening of the rock substance as they are wetted. Rockfill is likely to display a lower equivalent modulus than on previous drawdowns of the reservoir if the reservoir is drawn down below the previous lowest level, that is, the rockfill displays both over - and normally consolidated behaviour.

6.3.2 Methods of estimating the compressibility of earthfill, filters and rockfill

6.3.2.1 *Using data from the performance of other dams – earthfill*

Generally speaking, it will be sufficient to estimate the settlement of the embankment earthfill during and after construction, using the results of monitoring of other dams. G. Hunter, Hunter (2003) and Hunter and Fell (2003d), has gathered data from a large number of dams, where the core has been generally compacted to a minimum density of 98% of standard maximum dry density, with a water content in the range 2% dry to 1% wet of standard optimum water content. Most of the cores are 'dry placed', with a moisture content 0.5 to 1% dry of standard optimum moisture content.

(a) *Settlement during construction*

Figure 6.42 shows total settlements of the core during construction of earth and earth and rockfill embankments. These have been obtained from monitoring of cross arm settlement gauges embedded in the dam core near the dam centreline.

Tables 6.5 and 6.6 give the equations of best fit for the data.

(b) *Strains and equivalent moduli during construction*

Figure 6.43 and Table 6.7 shows the vertical strains versus effective vertical stress (ignoring suction effects) and estimated confined secant moduli in dry placed earthfill cores at the end of construction. The confined secant moduli (M) can be converted to Young's moduli using Equation 6.30. The Poisson's ratio is likely to be in the range 0.25 to 0.35 for dry placed earthfill.

(c) *Post construction settlement*

Figure 6.44 shows post construction crest settlements 10 years after the end of construction. Figure 6.45 shows long term post construction crest settlement rates for zoned earthfill and earth and rockfill embankments.

Table 6.8 shows typical ranges of post construction settlement and long term settlement rates for different core materials, core widths and compaction moisture content. These exclude 'outliers'.

The core width is classified as:

Thin – width <0.5 dam height.
Medium – width ≥ 0.5 and ≤ 1.0 dam height.

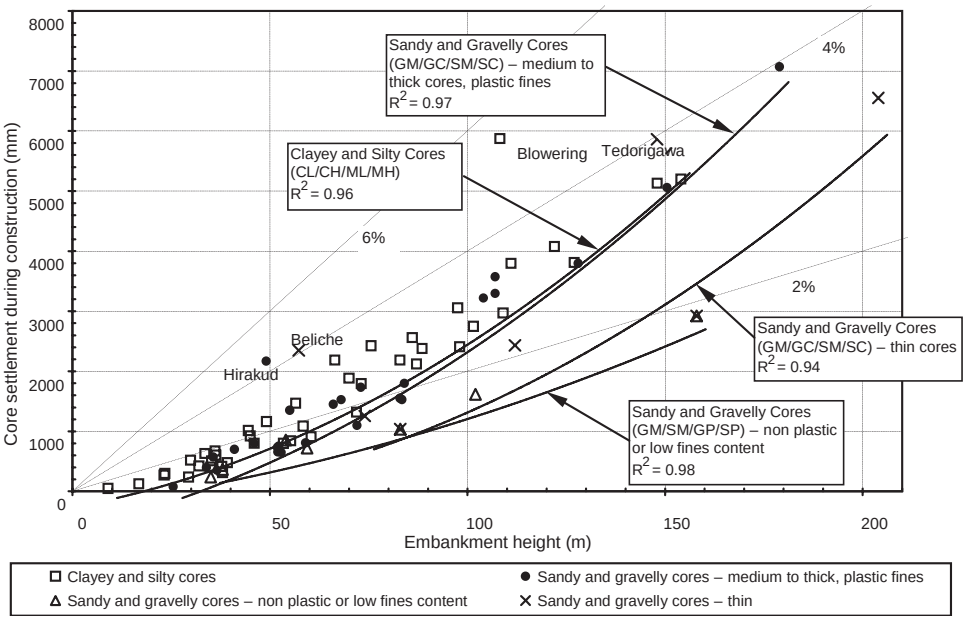


Figure 6.42 Total settlements during construction of earth and earth and rockfill embankments (Hunter, 2003, Hunter and Fell, 2003d).

Table 6.5 Equations of best fit for core settlement versus embankment height during construction (Hunter, 2003, Hunter and Fell, 2003d).

Core material/ shape type	*No. cases*	*Equation for settlement*1*	*R^2*2*	*Std. Err. of Settlement *3 (mm)*
Clay cores – all sizes	42	Settlement = H(0.152H + 12.60)	0.96	275
Sandy and gravelly cores:				
medium to thick, plastic	25	Settlement = H(0.183H + 7.461)	0.97	290
thin	5	Settlement = H(0.136H + 2.620)	0.94	635
non-plastic	7	Settlement = H(0.063H + 8.57)	0.98	130

Notes: *1 settlement in millimetres, embankment height *H* in metres.
*2 R^2 = regression coefficient.
*3 Std. err. = standard error of the settlement.

Wide – width >1.0 dam height.

Hunter (2003) and Hunter and Fell (2003) conclude:

For the crest region, the data has been sorted based on core width, core material type and placement moisture content, and indicates that:

– The post construction crest settlements are generally much smaller than the core settlement during construction.

Table 6.6 Equations of best fit for core settlement (as a percentage of embankment height) versus embankment height during construction (Hunter, 2003, Hunter and Fell, 2003).

Core material/ shape type	*No. cases*	*Equation for settlement**1 *(as a percentage of embankment height)*	R^2 *2	*Std. err. of settlement* *3 *(mm)*
Clay cores – all sizes	42	Settlement (%) = 0.179 $H^{0.60}$	0.77	0.41
Sandy and gravelly cores:				
medium to thick, plastic	25	Settlement (%) = 0.079 $H^{0.76}$	0.81	0.39
thin	5	Settlement (%) = 0.063 $H^{0.72}$	0.70	0.47
non-plastic	7	Settlement (%) = 0.187 $H^{0.46}$	0.71	0.24

Notes: *1 settlement as a percentage of the embankment height, embankment height H in metres.
*2 R^2 = regression coefficient.
*3 Std. Err. = standard error of the settlement (percent of embankment height).

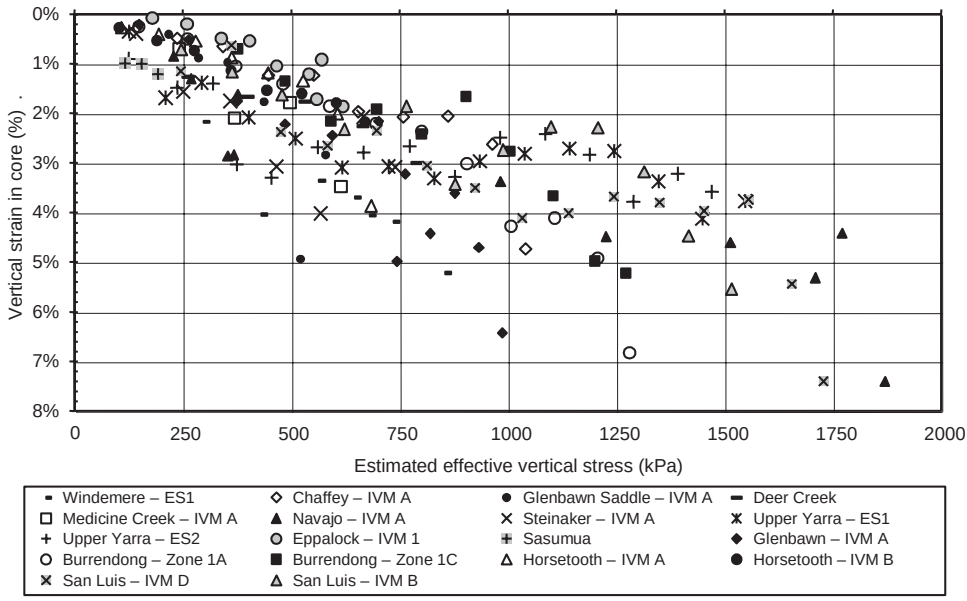


Figure 6.43 Vertical strain versus effective vertical stress in the core at end of construction for dry placed dominantly sandy and gravely earthfill (Hunter, 2003, Hunter and Fell, 2003d).

- Nearly all dams experience less than 1% crest settlement post construction for periods up to 20 to 25 years and longer after construction.
- Most experience less than 0.5% in the first 3 years and less than 0.75% after 20 to 25 years.
- Smaller magnitude settlements are observed for dry placed clayey sands to clayey gravels and dry to wet placed silty sands to silty gravels.
- A broader range of settlement magnitude is shown for clay cores, wet placed clayey sand to clayey gravel cores, and embankments with very broad core widths.

Table 6.7 Confined secant moduli during construction for well-compacted, dry placed earthfill. (Hunter, 2003, Hunter and Fell, 2003d).

		Confined secant modulus (MPa)[1] *Effective vertical stress range (kPa)*		
Core material type	*No. cases*	*500 to 700*[2]	*1000*	*1500 to 2000*
Clayey soil types (CL) – sandy clays and gravelly clays	20	15 to 45 (27)	15 to 50 (30)	25 to 42
Silty soils (ML), data from Gould.	3	30 to 60	–	–
Clayey Sands to Clayey Gravels (SC/GC), plastic fines, >20% fines	11	20 to 65 (33)	30 to 65 (40)	25 to 65
Silty Sands to Silty Gravels (SM/GM), plastic fines, >20% fines	9	35 to 65 (46)	45 to 65 (50)	35 to 65
Sandy and Gravelly soils – non-plastic fines or <20% plastic fines	6	35 to 80 (60)	35 to 90 (60)	40 to 90

Notes: *[1]Values represent range of confined secant moduli, values in brackets represent average.
*[2]Includes Gould (1953, 1954) data.

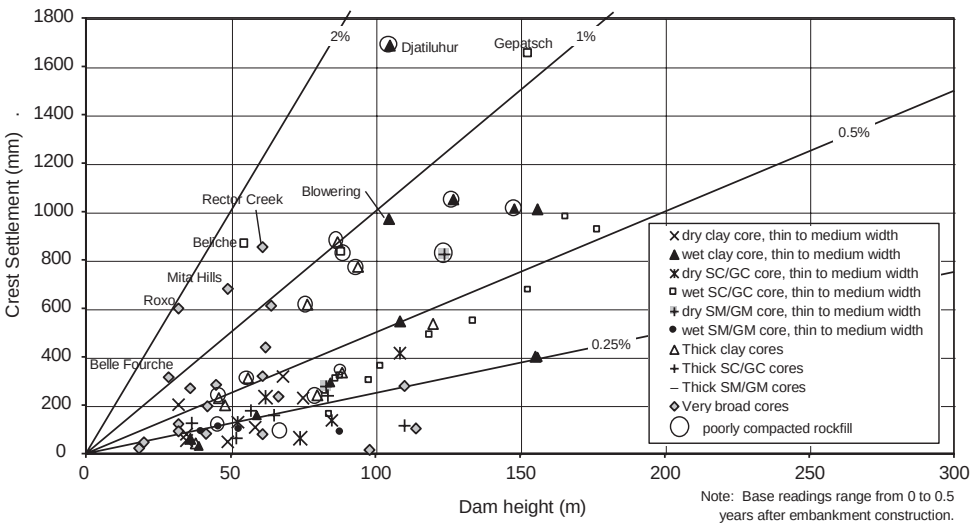


Figure 6.44 Post construction crest settlement at 10 years after end of construction (Hunter, 2003, Hunter and Fell, 2003d).

– For zoned earth and rockfill dams, poor compaction of the rockfill is over-represented for case studies at the larger end of the range of crest settlement.

The data for the shoulder regions indicates:

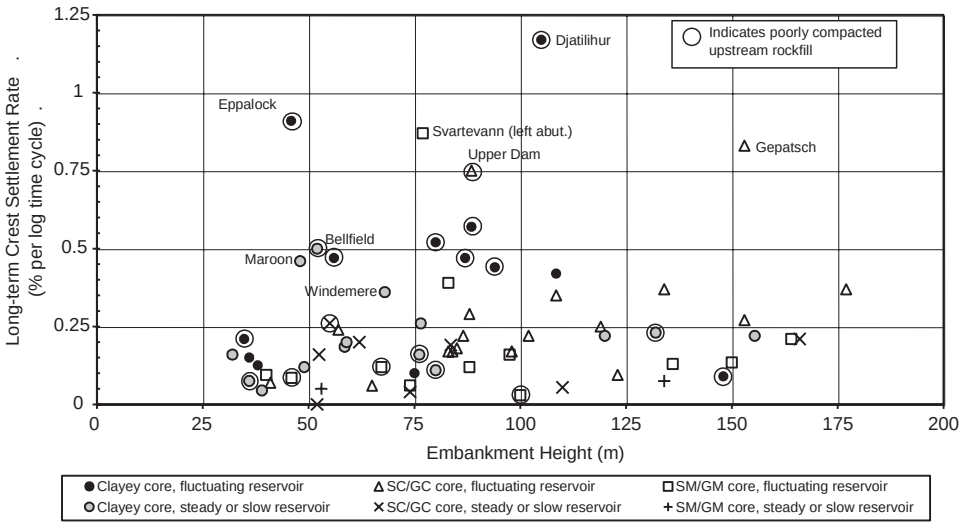


Figure 6.45 Long-term post construction crest settlement rate for zoned earthfill and earth-rockfill embankments of thin to thick core widths (Hunter, 2003, Hunter and Fell, 2003d).

Table 6.8 Embankment crest region, typical range of post construction settlement and long-term settlement rate. (Hunter 2003, Hunter and Fell, 2003d).

Core properties			*Crest settlement (%)* *1,*2			*Long-term settlement rate*1,*3*	
Classification	*Core width*	*Moisture content*	*3 years*	*10 years*	*20 to 25 years*	*Reservoir steady/slow*	*Fluctuating reservoir*
CL/CH	Thin to medium	dry	0.05 to 0.55	0.10 to 0.65	0.20 to 0.95	0.04 to 0.50	0.09 to 0.57
		wet	0.04 to 0.75	0.08 to 0.95	0.20 to 1.10	(most < 0.26)	
	Thick	all (most dry)	0.02 to 0.75	0.10 to 1.0	0.5 to 1.0		
SC/GC	Thin to medium	dry	0.10 to 0.25	0.10 to 0.40	<0.5	0 to 0.26	0.06 to 0.37
		wet	0.15 to 0.80	0.20 to 1.10	<1.1		
	Thick	all(most dry)	0.05 to 0.20	0.10 to 0.35	0.10 to 0.45		
SM/GM	Thin to thick	all	0.06 to 0.30	0.10 to 0.65	<0.5 to 0.7	<0.10	0.03 to 0.21
Very Broad Earthfill Cores – most CL and dry placed			0.0 to 0.60	0.0 to 0.80	0.05 to 0.76	0.08 & 0.44	0.07 to 0.70 (most < 0.35)

Notes: *1 excludes possible outliers.
*2 crest settlement as a percentage of the embankment height.
*3 long-term settlement rate in units of % settlement per log cycle of time (settlement as a percentage of dam height).

- Post construction settlements in the order of 1 to 2% are observed for poorly compacted rockfills. Greater settlements are observed for the dry placed, poorly compacted rockfill
- For reasonably compacted rockfill the range of settlement is quite broad, from 0.1% up to 1.0%, but the number of cases is limited. Settlements toward the upper range are observed for dry placed and/or weathered rockfills, where settlements due to collapse compression are likely to be significant.
- Much lower settlements, generally less than 0.5 to 0.7% at ten years after construction, are observed for well and reasonably to well compacted rockfill, and compacted earthfill.
- Very low settlements (less than 0.25% at 10 years) are observed for embankments with gravel shoulders.

6.3.2.2 *Using data from the performance of other dams – rockfill*

See Section 15.2.4.

6.3.2.3 *In situ testing*

The drained modulus E′ for free draining soils, such as sands, silty sands and fine gravelly sands, and the undrained modulus Eu for clayey soils can be estimated using Standard Penetration and Cone Penetration Tests.

Figures 6.46 and 6.47 show plots of E′ and Eu respectively versus SPT 'N' value. The moduli were obtained by pressuremeter testing. It will be seen that there is a considerable uncertainty in the relationship between the moduli and SPT 'N' value. Note that the moduli are in bars, where 10 bars = 1 MPa.

Figure 6.48 presents a chart to estimate the secant Young's modulus (E'_s) for an average axial strain of 0.1% for a range of stress histories and aging. This level of strain is reasonably representative for many well-designed foundations. In Figure 6.48a = 100 kN/m^2 and σ'_{mo} is the mean stress level = $(\sigma'_{vo} + 2\sigma'_{Ho})/3$. The stiffness of normally consolidated aged sands (>1000 years) appears to fall between that of very recent normally consolidated sands and over-consolidated sands.

Recent sands would include those deposited by dredging, or in historic time by for example scour and deposition in river channels.

Kulhawy and Mayne (1988) produced Figures 6.49 and 6.50 showing the relationship between $M/q_c(=\alpha_n)$ and relative density for normally consolidated (NC) and over-consolidated (OC) sands. As can be seen there is a trend for decreasing α_n with increase in relative density, but a large scatter of test data. Note that:

$$M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \tag{6.42}$$

where E′ = Young's modulus, M′ = constrained drained modulus, ν = Poisson's ratio, q_c = cone resistance.

Kulhawy and Mayne (1988) also discuss Poisson's ratio and suggest that for drained loading:

$$\nu' = 0.1 + 0.3\phi'_{rel} \tag{6.43}$$

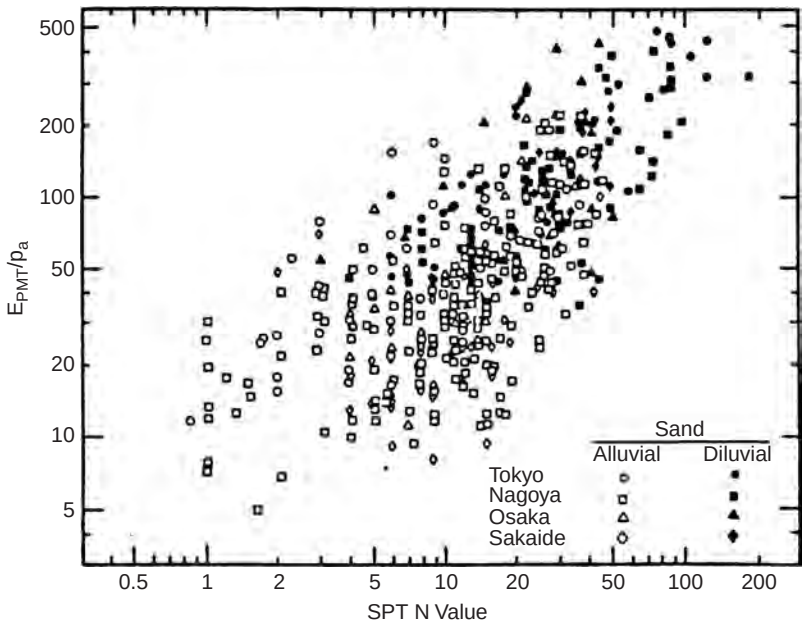


Figure 6.46 Drained Young's Modulus E′ for sands versus SPT 'N' value (Ohya *et al.*, 1982, from Kulhawy and Mayne, 1988).

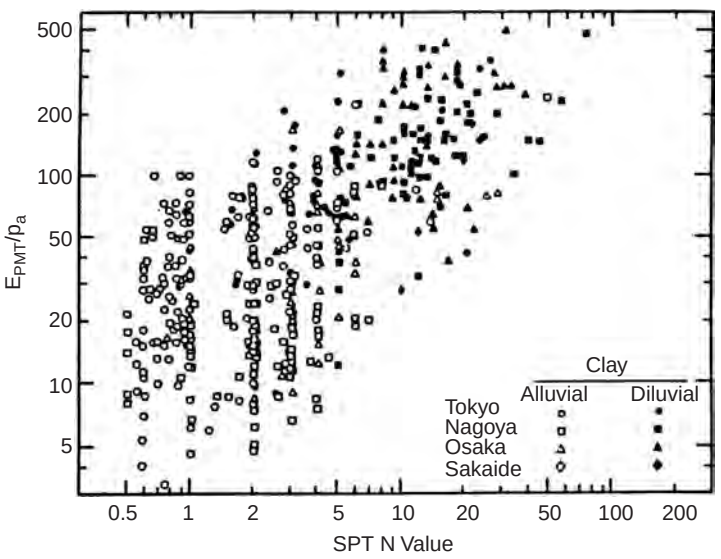


Figure 6.47 Undrained Young's Modulus E_u for clays vs SPT 'N' value (Ohya *et al.*, 1982, from Kulhawy and Mayne, 1988).

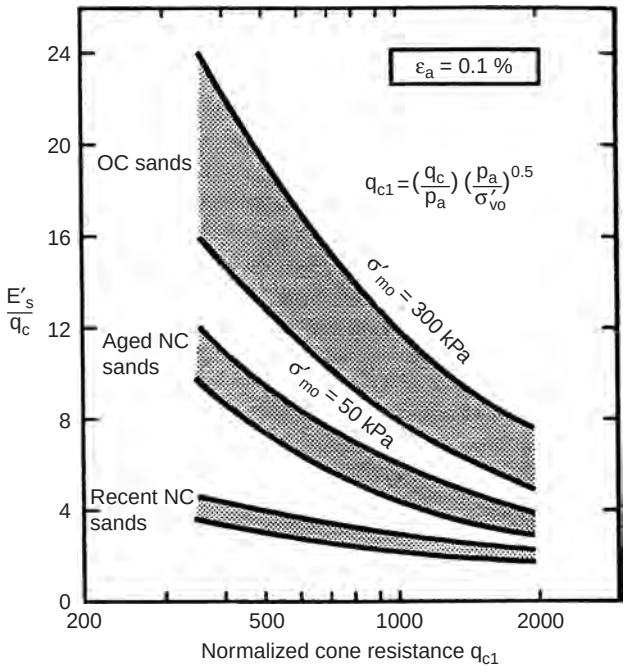


Figure 6.48 Drained Young's modulus E′ of sands versus Static Cone Penetrometer resistance (Bellotti *et al*., 1989, from Lunne *et al*., 1997).

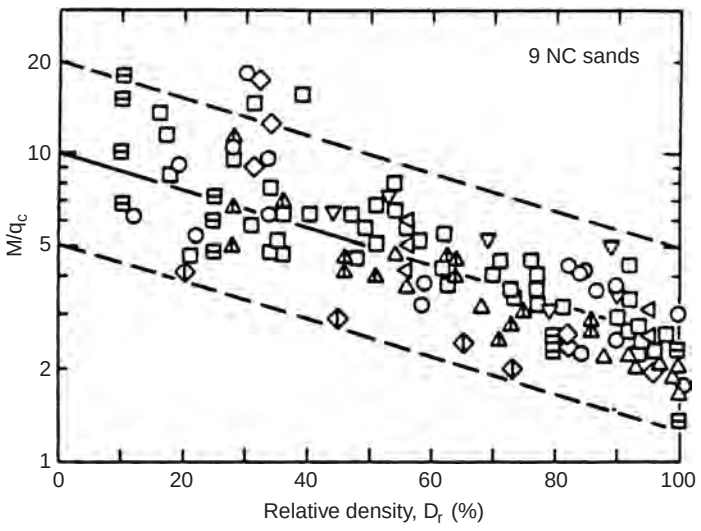


Figure 6.49 Variation of α_n (=M/q_c) with relative density for normally consolidated sand (Kulhawy and Mayne, 1988).

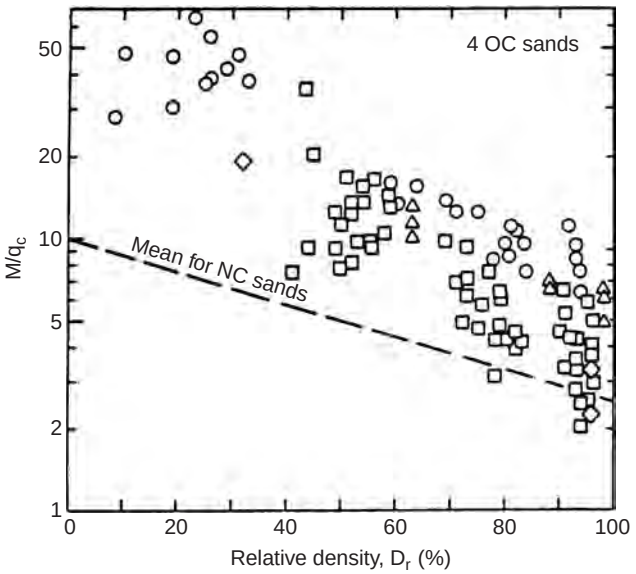


Figure 6.50 Variation of α_n ($= M'/q_c$) with relative density for overconsolidated sand (Kulhawy and Mayne, 1988).

where:

$$\phi'_{rel} = \frac{\phi' - 25^\circ}{45^\circ - 25^\circ} \tag{6.44}$$

with $0 \leq \phi'_{rel} \leq 1$, where ϕ' is the friction angle of soil at its relative density.

They suggest the use of Figure 6.51 (based on work by Wroth, 1975) to estimate ν' for lightly over-consolidated soils. For over-consolidated soils, Figure 6.52 (based on Poulos, 1978) may be used to adjust the values from Figure 6.53). In Figure 6.46 the stress level is the ratio of actual stress to failure stress.

In reality it is often very difficult to assess the degree of over-consolidation of sands, unless they are overlain by clays for which the OCR can be determined by laboratory oedometer tests, or the erosion/depositional history of the site can be established. In other cases it is usually necessary to assume the soil is normally consolidated which may give conservatively low moduli.

The undrained Young's Modulus E_u for cohesive soils cannot be estimated with any degree of accuracy directly from q_c values. E_u can be estimated indirectly from CPT or CPTU, by first estimating S_u, and then relating E_u to S_u by using Figure 6.53.

The Self Boring Pressuremeter (SBPM) can be used to determine the drained modulus in sandy soils and undrained modulus in clayey soils. Details are given in Clayton *et al.* (1995).

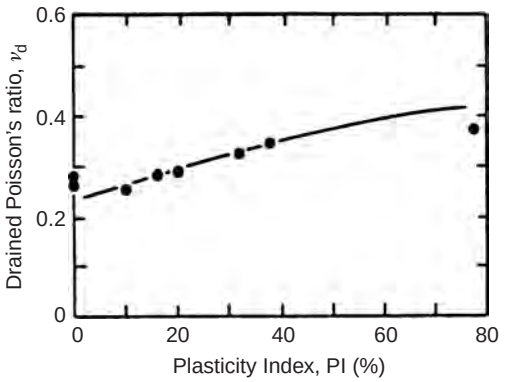


Figure 6.51 Drained Poisson's ratio ν' vs plasticity index for some lightly over-consolidated soils (Kulhawy and Mayne, 1988, and Wroth, 1975).

The SBPM test gives the most reliable means of determining the modulus, but is relatively expensive. Usually it would only be used on important projects, and then often as a means of calibrating CPT tests.

The Menard Type Pressuremeter is inserted into a pre-drilled borehole. It is very dependent on the drill hole staying open and is affected by disturbance, particularly in sands and weak clays. More recent developments of the Menard Pressuremeter include a push-in pressuremeter and a self-boring type. These should have a similar accuracy to the SBPM.

6.3.2.4 *Laboratory testing*

The use of laboratory triaxial tests to determine the modulus of a soil is discussed in Baldi *et al.* (1988), Jardine *et al.* (1984), Atkinson and Evans (1984). They point out that conventional triaxial equipment is not able to measure the modulus and, if used, will generally significantly underestimate the modulus. Generally it will be better to rely on estimated moduli, or values determined by in situ testing as described above.

6.3.2.5 *Tensile properties of plastic soils*

The conventional understanding in embankment dam engineering is that if the core of a dam is compacted dry of standard optimum moisture content the soil will be "brittle", and that if it is compacted wet of standard optimum moisture content it will be more flexible and less likely to crack.

Win (2006) carried out direct tensile strength tests on several soils and found that:

(1) The tensile strength is related to the compaction water content as shown in Figure 6.54. Soils compacted dry of standard optimum water content have higher tensile strengths than the same soil compacted wet of optimum.
(2) The tensile strain at failure was independent of the compaction water content for the two clay soils (soils C and D), but was influenced by the compaction water content for the silty sand soil F (Figure 6.55).

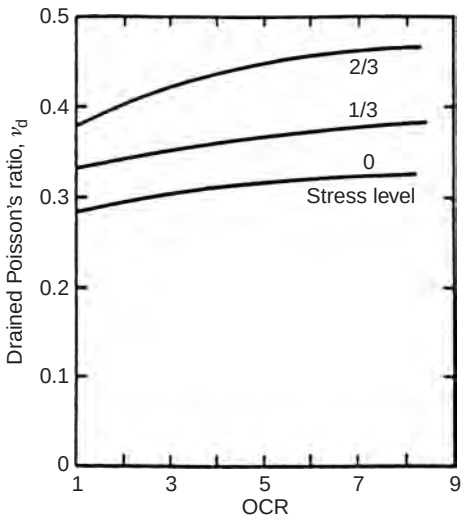


Figure 6.52 Drained Poisson's ratio ν' versus OCR and stress level for clay (Kulhawy and Mayne, 1988, and Poulos, 1978).

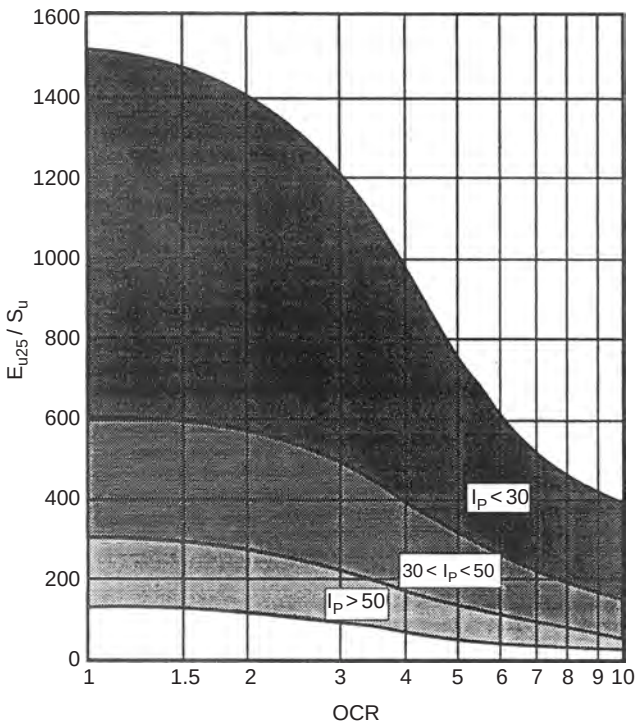


Figure 6.53 Relationship between undrained modulus at a stress ratio of 25% of the ultimate strength (Eu_{25}) versus over-consolidation ratio (OCR), and plasticity index (PI) (Lunne *et al.*, 1997, after Duncan and Buchignani, 1976).

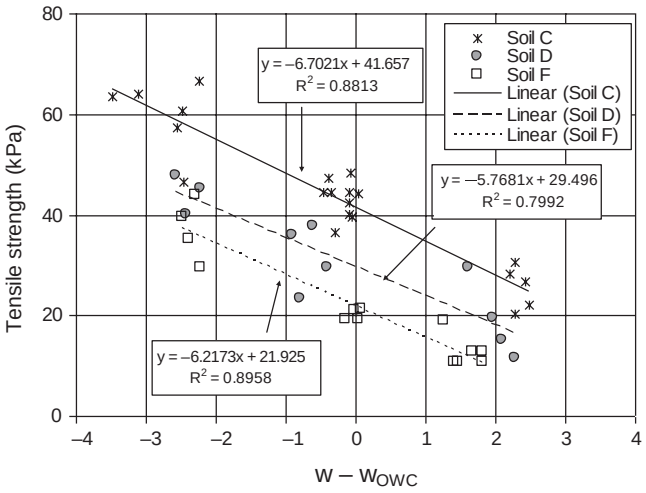


Figure 6.54 Effect of compaction water content on tensile strength at failure (Win, 2006).

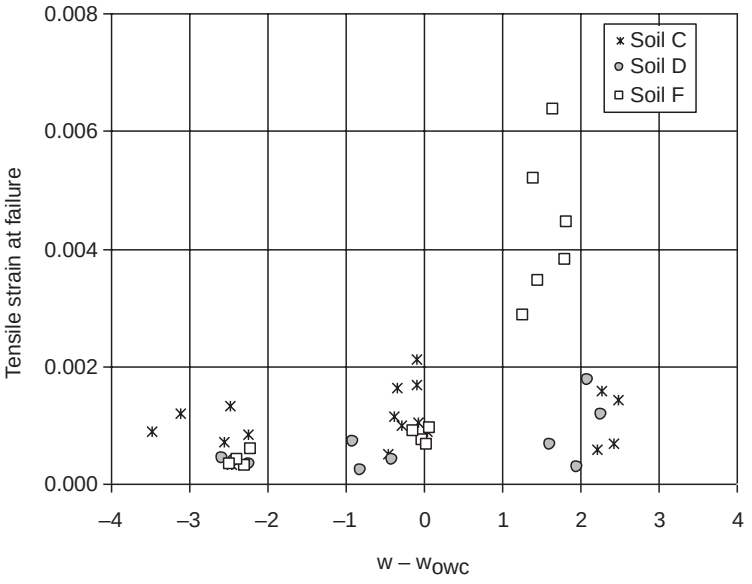


Figure 6.55 Effect of compaction water content on tensile strain at failure (Win, 2006).

(3) Soils compacted dry of optimum had higher moduli than those compacted at or wet of optimum water content (Figure 6.56).
(4) There is an increase in tensile strength for clay soils compacted between 95% and 98% standard maximum dry density but no increase above that level.

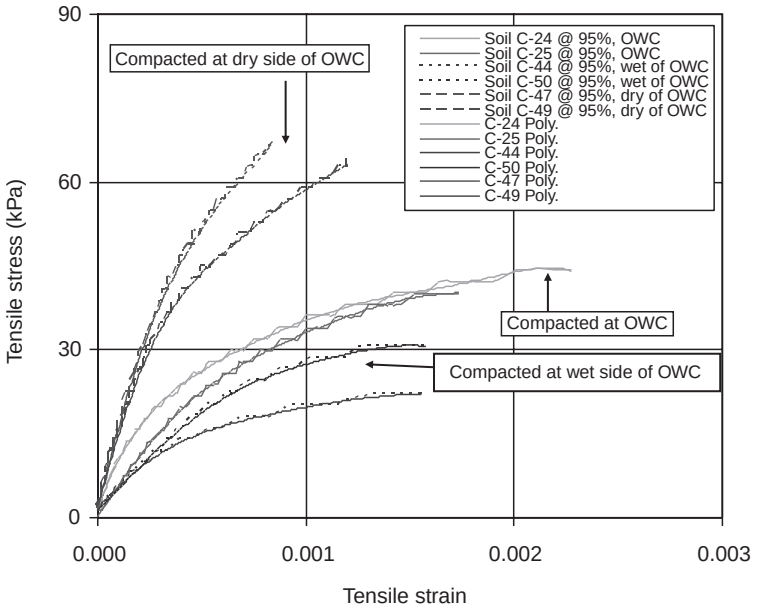


Figure 6.56 Effect of compaction water content on stress-strain behavior (Win, 2006).

These findings generally support the conventional understanding in regards to "brittleness". But the situation is more complex in regards to whether cracking will occur. As the tests by Win (2006) show lower tensile strength for soils compacted wet of optimum and similar tensile strains at failure on the surface compacting wet of optimum would make the soil more susceptible to cracking wet of optimum. However for cross valley analysis of deformations as discussed in Sections 8.3.2 and 11.7.2 the most critical parameter determining the depth and magnitude of tensile stresses and strains is the Poisson's ratio. Soils compacted wet of optimum are likely to have a higher Poisson's ratio than soils compacted dry of optimum given the same degree of compaction because the voids are more saturated. Higher Poisson's ratio gives less lateral strain and lower tensile stresses and strains in cross valley deformation analyses and hence less likely for cracking to occur.

6.4 PERMEABILITY OF SOILS

6.4.1 General principles

The 'permeability', or more correctly 'permeability coefficient' or 'hydraulic conductivity' of the soil in a dam embankment or foundation, is not a fundamental property of the soil but depends on a number of factors. Head (1985) outlines these as: particle size distribution; particle shape and texture; mineralogical composition; void ratio; degree of saturation; soil fabric; nature of fluid; type of flow and temperature.

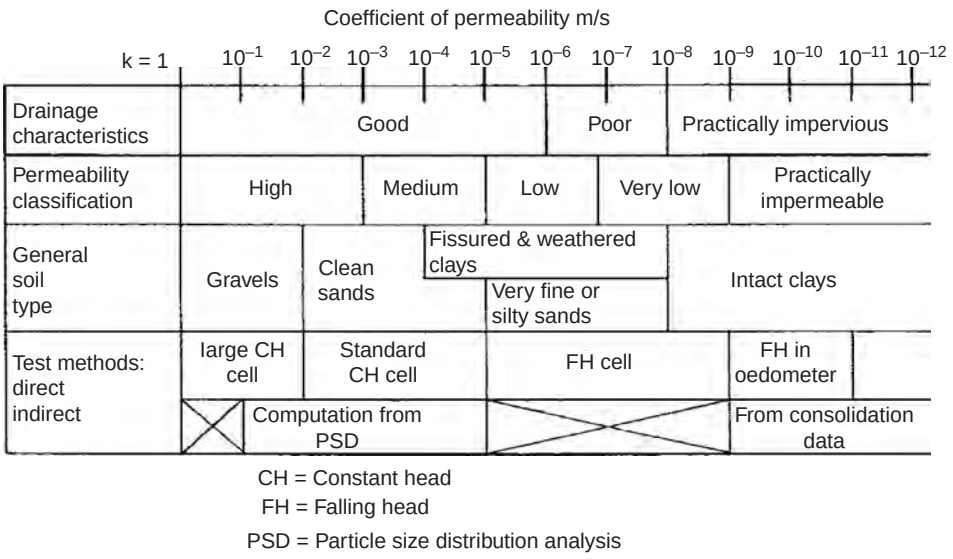


Figure 6.57 Permeability and laboratory testing method for the main soil types (Head, 1985).

In embankment dam engineering these factors have varying degrees of influence:

Particle size distribution. The permeability is dependent on the particle size distribution of the soil with fine grained clay soils having permeabilities which are several orders of magnitude lower than that of coarser soils, i.e. sands and gravels. Figure 6.57 shows in general terms the range of permeabilities which can be encountered.

It has been recognised that the finer particles in a soil largely determine its permeability, and granular soils are often compared by their 'effective grain size', D_{10}, which is the particle size for which 10% of the soil is finer.

Particle shape and texture. This affects permeability to a lesser extent. Elongated ('platy') particles tend to have a lower permeability than rounded, and rougher textured particles a lower permeability than smooth.

Mineralogical composition. This is a factor in clay soils. Montmorillonite clays, for example, are finer grained and have a greater tendency to adsorb water (and hence a lower permeability) than say a kaolin clay. The mineralogy of sand and gravel has little effect except where it results in elongated particles.

Void ratio. The void ratio of a soil has an important effect on permeability. Cohesive soils which are compacted to a high density ratio (e.g. 98% of standard maximum dry density) will have lower permeability than those compacted to a low density ratio (e.g. 90%). The difference may be orders of magnitude. Figure 6.58 shows the effect of void ratio for several clays.

The void ratio also has an effect on the permeability of granular soils, with soils compacted to a small void ratio (dense) having lower permeabilities than those with a high void ratio (loose). This effect is allowed for in the Kozeny-Carman formula discussed below.

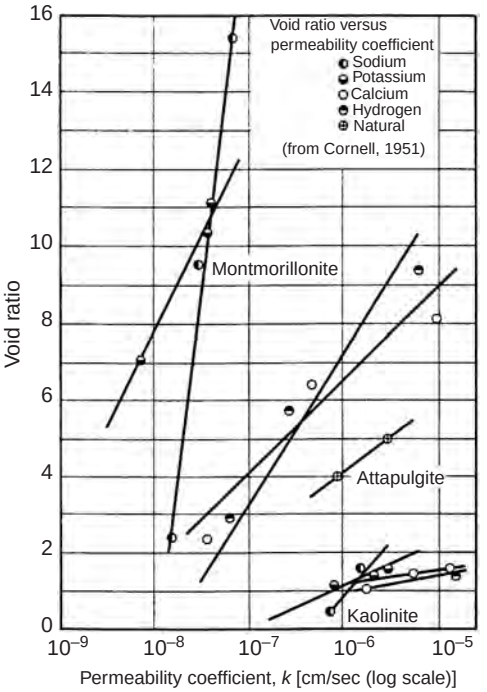


Figure 6.58 Effect of void ratio and clay mineralogy on permeability (Lambe and Whitman, 1981). Reprinted by permission of John Wiley & Sons Inc.

Degree of saturation. When a soil becomes partially saturated, the permeability is reduced. This occurs due to a reduction in the total cross section of pores filled with water and surface tension effects. These water filled pores tend to be the finer pores, because water is most readily removed from the larger pores which have low suction potential. The remaining finer water filled pores have naturally lower permeability. The partially saturated permeability may be an order of magnitude lower than the saturated permeability. Richards (1974) and Friedlund and Rahardjo (1993) discuss the relationship between saturated and unsaturated permeability for granular and cohesive soils.

Soil fabric. The fabric of a soil can have a major effect on the permeability. This is particularly important for soil masses *in situ*, where stratification or layering of different soil types, e.g. sand, silt and clay can lead to markedly different permeability along the strata to that across the strata. Ratios of horizontal to vertical permeability (K_H/k_v) of 100 or more are not uncommon.

Relict jointing, fissures, root holes, worm holes, lateritisation etc. all influence the soil fabric and lead to much higher permeability for the soil mass than the soil substance permeability.

When testing samples from such soil masses in the laboratory, it is essential that the presence of fabric is allowed for, e.g. in orienting samples along or across the strata

and in taking sufficiently large samples to include representative fissures. Rowe (1972) suggests that samples of up to 250 mm diameter may be needed to correctly sample fabric. Generally speaking if good quality *in situ* testing can be carried out, it will be more reliable than laboratory testing.

The permeability of recompacted soil is also affected by the soil fabric, which is dependent on the method of sample preparation. Lambe and Whitman (1981) describe this as micro-structure or micro-fabric effects. Figure 6.59 shows the effect of mixing of the soil before compaction, and of adding a dispersant (0.1% polyphosphate) ('complete mixing') to the soil. In the latter the dispersant breaks up the flocculated structure, resulting in a lower permeability.

The water content at which clay is compacted affects its permeability. In particular a cohesive soil compacted dry of optimum results in a less oriented structure, with potentially aggregates of soil separated by voids, leading to relatively high permeability. An example of this behaviour is given in Figure 6.60 where soils are compacted with the same compactive effect at different water contents.

It is virtually impossible to simulate in the laboratory the effect of different degrees of compaction at the top and bottom of layers and other factors, such as desiccation at the surface of layers during construction. In the dam, the ratio of kH/kV is likely to be high, possibly as high as 100, with the laboratory permeability probably best representing the vertical permeability kV.

Nature of the permeating fluid. The permeability (coefficient of permeability k) is dependent on the properties of the permeating fluid. It is related to the absolute (also known as intrinsic, or specific) permeability, K, by:

$$k = \frac{K\rho g}{\mu} \tag{6.45}$$

where ρ = fluid density, g = acceleration due to gravity, μ = fluid viscosity.

For most dam projects water is the permeating fluid and the small variations in viscosity and density, resulting from differences in temperature and dissolved salts content, are not significant compared to other variables. However the chemical composition of the permeating water can affect soil dispersivity (see Chapter 7) and to a certain extent also the permeability.

Type of flow. The basic assumption in calculations using Darcy's Law $q = kiA$ is that the soil is saturated and the flow is laminar. This is generally the case for flow through and beneath dams but flow in medium and coarse gravels may become turbulent in some cases, e.g. around dewatering wells. Flow through rockfill will generally be turbulent.

6.4.2 Laboratory test methods

Permeameters

Most permeability testing of soils is carried out using constant head or falling head permeability apparatus. As shown in Figure 6.61 the constant head test is best suited to high to medium permeability soils, sand and gravel, and the falling head test to low to very low permeability soils, silts, fine sands and some clays.

For very low permeability soils, the flow rates are too low to carry out testing in a conventional permeameter, and it would be more usual to do the testing in an

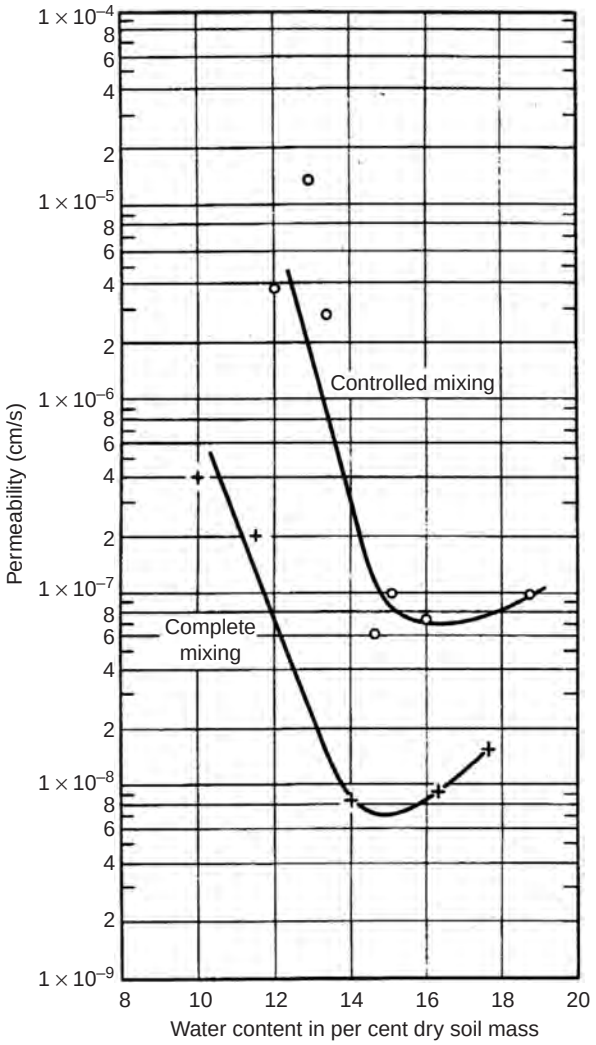


Figure 6.59 Effect of structure on permeability (Lambe and Whitman, 1981). Reprinted by permission of John Wiley and Sons Inc.

oedometer cell (e.g. a Rowe cell), or to determine the permeability indirectly from consolidation test data.

Many texts in soil mechanics include details of falling head and constant head tests so they are not repeated here. Head (1985) gives detailed instructions on laboratory procedures, as do Standards Association of Australia (1980) and American Society for Testing and Materials D2434. USBR (1985) also gives detailed instructions for the test in their "*Earth Manual*".

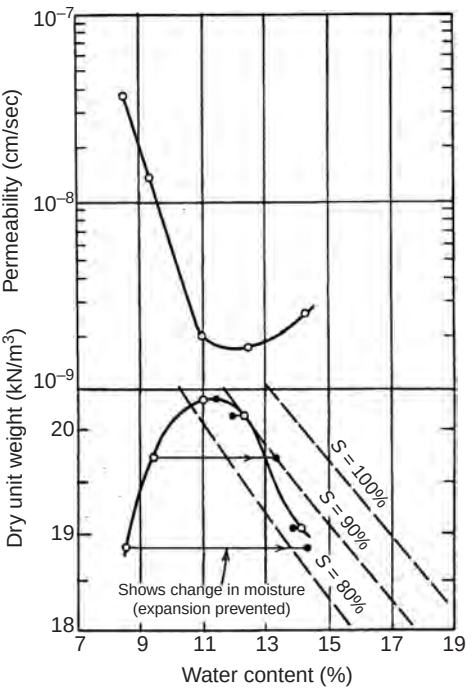


Figure 6.60 Effect of compaction water content on permeability (Lambe and Whitman, 1981). Reprinted by permission of John Wiley and Sons Inc.

Some common errors and problems which can arise with permeability testing include:

(a) Leakage past the sample in the test mould.
(b) Variability in test results for compacted soils due to difficulty in maintaining uniform water content and density in sample preparation.
(c) Failure to saturate sample.
(d) Failure to adequately test the fabric of the soil.
(e) Testing under little or no confining stress. For dams work in particular it is important to use apparatus such as that shown in Figure 6.62 which allows simulation of the overburden pressure on the soil and in so doing allows testing at the correct void ratio.
(f) Excessive flow rates in the apparatus (when testing in coarse sand or gravel) leading to unaccounted losses in the pipe work or porous plates at the ends of the sample and an underestimation of permeability.

From the above, a) and c) can be eliminated by careful test procedures; b) is to be expected and requires several tests so that variability can be identified; and d) can be overcome by correct sampling, using larger samples and/or using *in situ* tests. Problems with high flow rates, f) can be readily checked by testing the apparatus without soil.

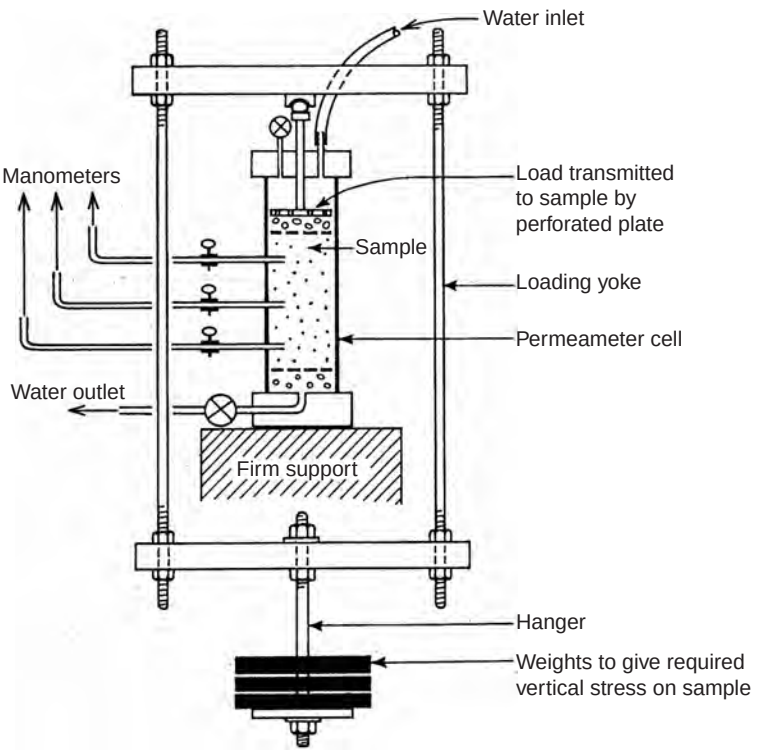


Figure 6.61 Suitable assembly for constant head permeability apparatus (Head, 1985).

6.4.3 Indirect test methods

6.4.3.1 *Oedometer and triaxial consolidation test*

For very low permeability clays, it is often more accurate to estimate the permeability from the results of oedometer consolidation tests, rather than to carry out falling head tests. From the theory of consolidation:

$$k = \gamma_w c_v m_v \tag{6.46}$$

where γ_w = unit weight of water, c_v = coefficient of consolidation, m_v = coefficient of volume change.

In any oedometer tests c_v and m_v commonly depend on the pressure applied to the soil, so the permeability k also varies depending on the applied pressure. If a Rowe consolidation cell is used with radial drainage (rather than vertical), the horizontal coefficient of consolidation and hence horizontal permeability will be calculated. Details are given in Head (1985).

Similarly, the consolidation phase of a triaxial test can be used to estimate m_v and c_v and hence the permeability. If radial drainage is promoted by surrounding the

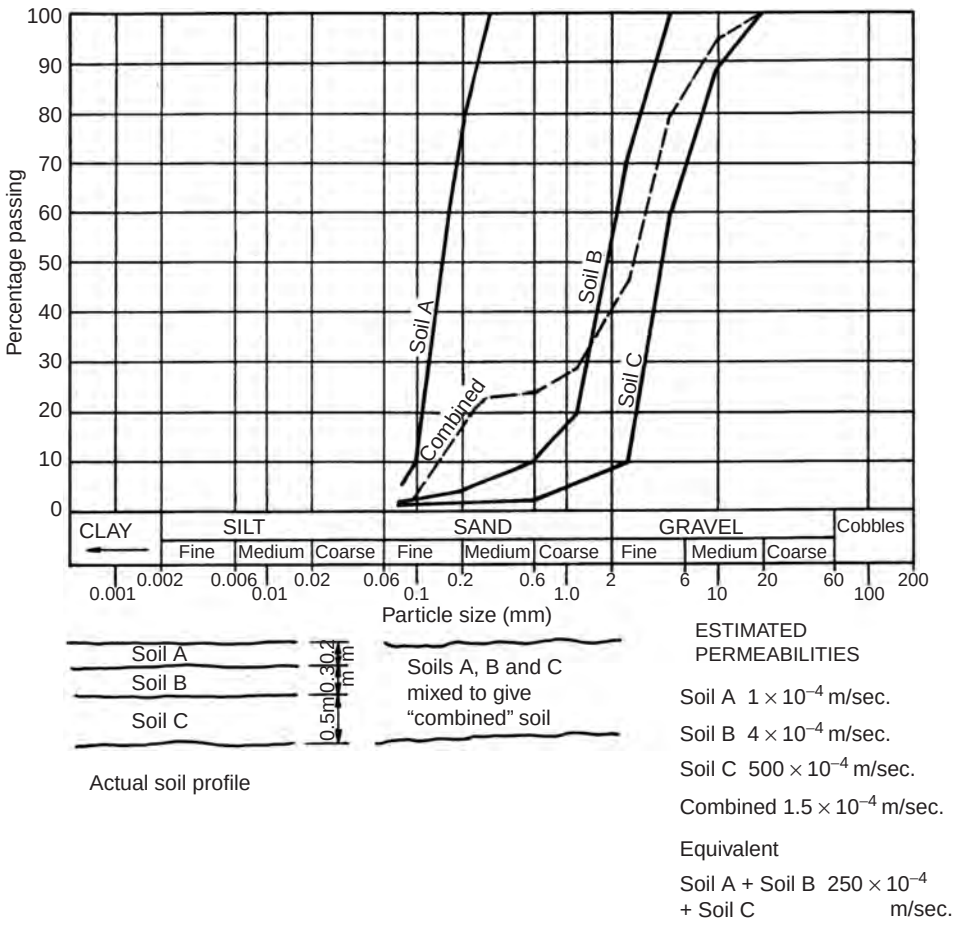


Figure 6.62 Effect of mixing of soil on estimated permeability.

sample with filter paper, the horizontal coefficients of consolidation, volume change and permeability will be determined.

An advantage of using triaxial testing is that saturation of the sample can be assured by back pressure saturation. However the permeability of the filter paper drains may control the flow from the sample (Germaine and Ladd, 1988), so care must be taken in using them.

For many soils the presence of gravel particles makes it impractical to carry out triaxial tests so *in situ* tests on the soil in the embankment must be used.

6.4.3.2 *Estimation of permeability of sands from particle size distribution*

An approximate estimate of the permeability of sands and sandy gravels can be obtained from the particle size distribution of the soil. The most commonly used of the available formulae are:

(i) *Hazen's formula*. The most commonly quoted form of the equation is

$$k = C(D_{10})^2 \tag{6.47}$$

where C = factor, usually taken as = 0.01, D_{10} = effective grain size in mm, k = permeability in m/sec.

It should be noted that Hazen's formula was developed for uniform clean sands (less than 5% passing 0.075 mm) with D_{10} sizes between 0.1 mm and 3.0 mm. Even within these constraints C is quoted to vary from 0.004 to 0.015 (Holtz and Kovacs, 1981; Head, 1985). Lambe and Whitman (1979) show values of C varying from 0.01 and 0.42 with an average of 0.16 for a range of soils from silt to coarse sand. It is recommended that if the formula is used, an appropriate degree of conservatism be applied to the value of C selected. Depending on the particular circumstances, conservatism may involve selection of a low or high value of C depending on the application. Hazen's formula is NOT applicable to clays or to coarse gravels.

(ii) *Kozeny-Carman formula*. Head (1985) discusses the application of the Kozeny-Carman formula and suggests the form

$$k = \frac{2}{fS^2}\left(\frac{e^3}{1+e}\right) \text{ m/sec} \tag{6.48}$$

$$\text{where } S = \text{specific surface} = \frac{6}{\sqrt{d_1/d_2}} \tag{6.49}$$

and d_1, d_2 = maximum and minimum sized particles in mm, e = void ratio, f = angularity factor = 1.1 for rounded grains, for 1.25 for sub-rounded grains, 1.4 for angular grains.

This equation attempts to account for the effect of particle size, void ratio and particle shape or permeability.

It is unlikely that the Kozeny-Carman formula is significantly more accurate than the Hazen formula and its use should be limited to initial estimates of permeability of sand in non-critical projects.

(iii) *Sherard et al. (1984a) formula*. Sherard *et al.* (1984a) carried out laboratory permeability tests on 15 different filter sands and sandy gravels with D_{15} in the range from 0.1 to 10 mm. The materials were mostly broadly graded and with little or no fines.

From these tests they concluded that the permeability could be estimated from:

$$k = C(D_{15})^2 \tag{6.50}$$

where: k = permeability in metres/sec, D_{15} = particle size for which 15% is finer in mm, C = 0.0035 with a range of median test values of C = 0.002 to C = 0.006 for densely compacted filters (70% relative density.

Sherard *et al.* (1984a) found that the permeability of the same materials placed very loose were higher by a ratio in the range 1.1 to 3.0 but up to 13.7 for a very broadly graded silt sand gravel soil.

6.4.4 Effects of poor sampling on estimated permeability in the laboratory

Many alluvial soils are stratified *in situ*, e.g. a sandy gravel/gravelly sand deposit in a river bed will, in fact, consist of separate beds of fine sand, coarse sand, and gravel.

When sampled in boreholes or test pits, particularly below the water table, there is often a tendency to mix the strata, in the drilling process and/or in the sampling. If these mixed samples are used for laboratory permeability tests, or their permeability is inferred from their particle size distribution, it is likely that the permeability for flow along the strata, i.e. essentially horizontal permeability, will be significantly underestimated.

Figure 6.62 gives an example where soils A, B and C are present as distinct layers. Their approximate individual permeability values estimated using Hazen's formula are as shown for horizontal flow. Soil C (gravel) dominates the conditions, and the equivalent horizontal permeability is of the order of 250×10^{-4} m/sec.

If in the drilling and sampling process the soils are mixed in proportion to their thicknesses, the combined soil particle size distribution would be as shown. For the combined soil the permeability is controlled by the finer particles, resulting in an estimated permeability of 1.5×10^{-4} m/sec, or only 0.6% of the actual field horizontal permeability. This can only be overcome by very careful drilling and sampling, or preferably by *in situ* permeability testing.

6.4.5 *In situ* testing methods

In situ permeability testing in soil is described in Section 5.13. In general pump-out *in situ* tests, are the preferred method of estimating permeability, because the effects of the soil structure and fabric are properly accounted for. Pump-in *in situ* tests are potentially inaccurate as described in Section 5.13.

Chapter 7

Clay mineralogy, soil properties, and dispersive soils

7.1 INTRODUCTION

The properties of soils are determined by the properties of the constituent soil particles, the nature and quantity of water in the soil, the past consolidation history of the soil and soil structure. In this chapter the influence of the mineralogy of clays present in fine grained soils is discussed, with a particular emphasis on dispersive soils, and their use in the construction of embankment dams.

Dispersive soils are those soils which by the nature of their mineralogy and the chemistry of the water in the soil, are susceptible to separation of the individual clay particles and subsequent erosion of these very small particles through even fine fissures or cracks in the soil under seepage flows.

This is distinct from other highly erodible soils, such as silt and sand, which erode by physical action of the water flowing through or over the soil.

As discussed in Section 8.3.4.6 non-dispersive clays erode by a combination of physical action and detachment by slaking depending on their clay mineralogy.

For many years it has been recognised (e.g. Aitchison *et al.*, 1963; Aitchison and Wood, 1965) that the presence of dispersive soils either in the soil used to construct a dam, or in the dam foundation, greatly increases the risk of failure of the dam by "piping failure", i.e. development of erosion to the extent that a hole develops through the embankment, with rapid loss of water from the storage. Figure 7.1 shows such a case.

An understanding of the basic concepts of clay mineralogy is essential to the understanding of identification and treatment of dispersive soils in dam engineering. The following gives an outline of the subject. For more details the reader is directed to Mitchell (1976, 1993), Mitchell and Soga (2005) and Grim (1968). Holtz and Kovacs (1981) give a useful summary of the topic.

7.2 CLAY MINERALS AND THEIR STRUCTURE

7.2.1 Clay minerals

The basic "building blocks" of clay minerals are silica tetrahedra and aluminium (Al) or magnesium (Mg) octohedra. These give sheet like structures as shown in



Figure 7.1 Examples of piping failure of a dam constructed of dispersive soils (Soil Conservation Service of NSW).

Figure 7.2. The alumina octohedra are known as gibbsite, the magnesium octohedra are brucite.

These in turn combine to give the clay minerals. Figure 7.3 shows the structure of montmorillonite and kaolin.

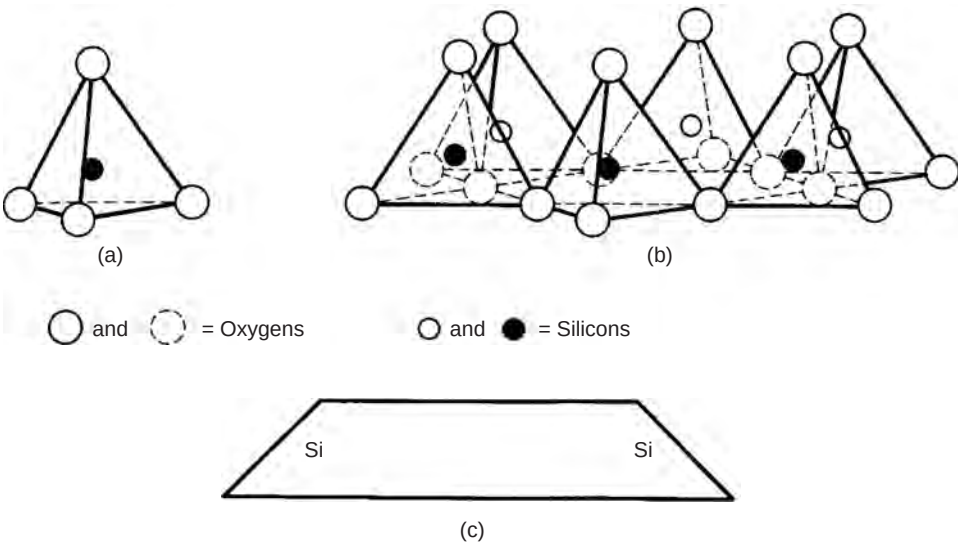
Some silicate clay minerals do not have a crystalline structure, even though they are fine grained and display clay-like engineering properties. These are known as allophane and are present in most soils. Mitchell (1976) indicates they are particularly common in some soils formed from volcanic ash because of the abundance of "glass" particles.

Oxides also occur widely in soils and weathered rock as fine-grained particles which exhibit claylike properties.

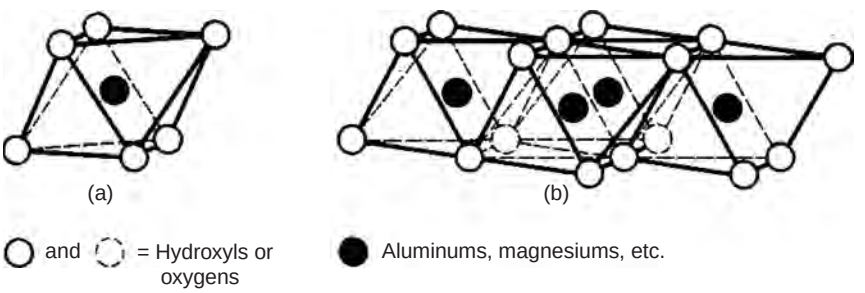
Examples are:

- Gibbsite, boehmite, haematite and magnetite (oxides of Al, Fe and Si which occur as gels, precipitates or cementing agents),
- Limonite: amorphous iron hydroxide,
- Bauxite: amorphous aluminium hydroxide.

They are common in tropical residual soils and soils derived from volcanic ash.



(a) Single silica tetrahedron, (b) Isometric view of the tetrahedral or silica sheet, (c) Schematic representation of the silica sheet.



(a) Single aluminium (or magnesium) octahedron, (b) Isometric view of the octahedral sheet, (c) Schematic representation of the octahedral or alumina (or magnesia) sheet.

Figure 7.2 Silica tetrahedra, and aluminium and magnesium octohedra (Holtz and Kovacs, 1981).

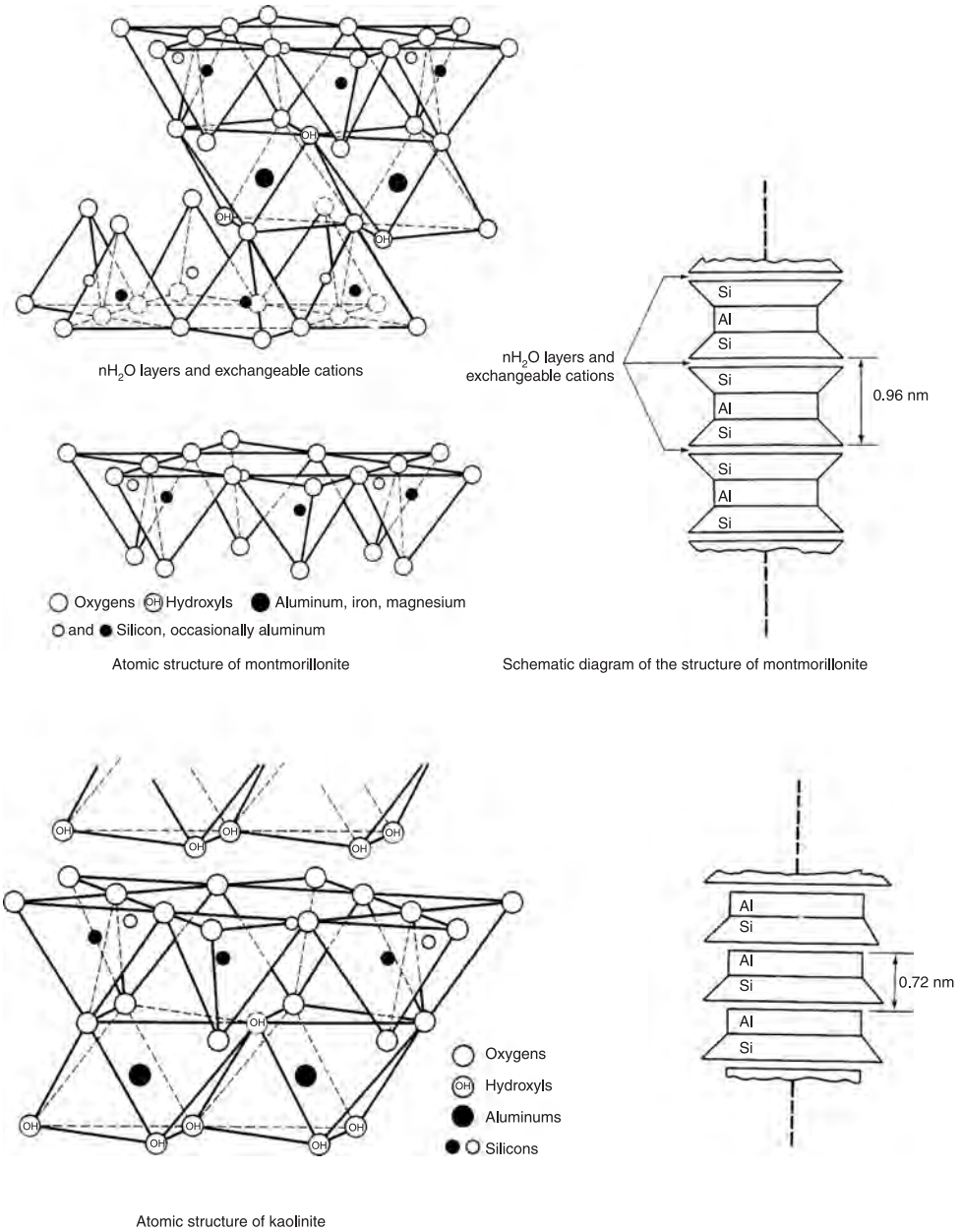


Figure 7.3 Structure of montmorillonite and kaolinite (Holtz and Kovacs, 1981).

7.2.2 Bonding of clay minerals

The properties of clays, particularly in a soil-water environment, are determined by the nature of the atomic bonding between the atoms which make up the sheets of clay

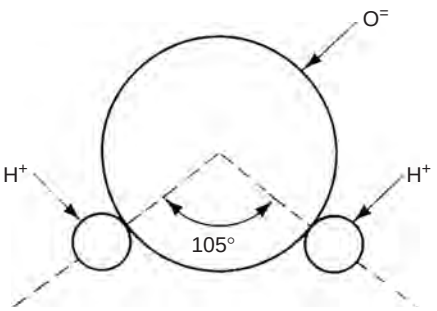


Figure 7.4 Schematic diagram of a water molecule showing its dipole nature (Holtz and Kovacs, 1981).

mineral, and the bonding between the sheets of clay mineral. The bonding mechanisms which are present include.

7.2.2.1 *Primary bonds*

These are either:

(i) *Covalent bonds*, where two atomic nuclei share one or more electrons, e.g. $H\bullet + \bullet H = H{:}H$, or the hydrogen molecule

(ii) *Ionic bonds*, which result from the electrostatic attraction between positive and negative ions (ions are free atoms which have gained or lost an electron). Positive charged ions are known as cations, negatively charged as anions.

When an ionic bond forms, the centres of negative and positive charge are separated and form a dipole. The dipole then has a positive and negatively charged "end", even though it is neutral overall. For example, we have common salt.

$$Na^+ + Cl^- = NaCl$$

Water is another example of a molecule which forms a dipole as shown in Figure 7.4. This dipolar nature has an important effect on the behaviour of water in clays.

Mitchell (1976) indicates that purely ionic or covalent bonds are not common in soils, and a combination is more likely. Silica (SiO_2) is partly ionic, partly covalent.

(iii) *Metallic bonds*, which are of little importance in the study of soils.

7.2.2.2 *Secondary bonds*

(i) *Hydrogen bonds*. Attraction will occur between the oppositely charged ends of permanent dipoles. When hydrogen is the positive end of the dipole, the resulting bond is known as hydrogen bonding.

(ii) *Van der Waal's bonds*. As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. This will not be permanent in one direction. The attraction of the positive and negatively charged ends of

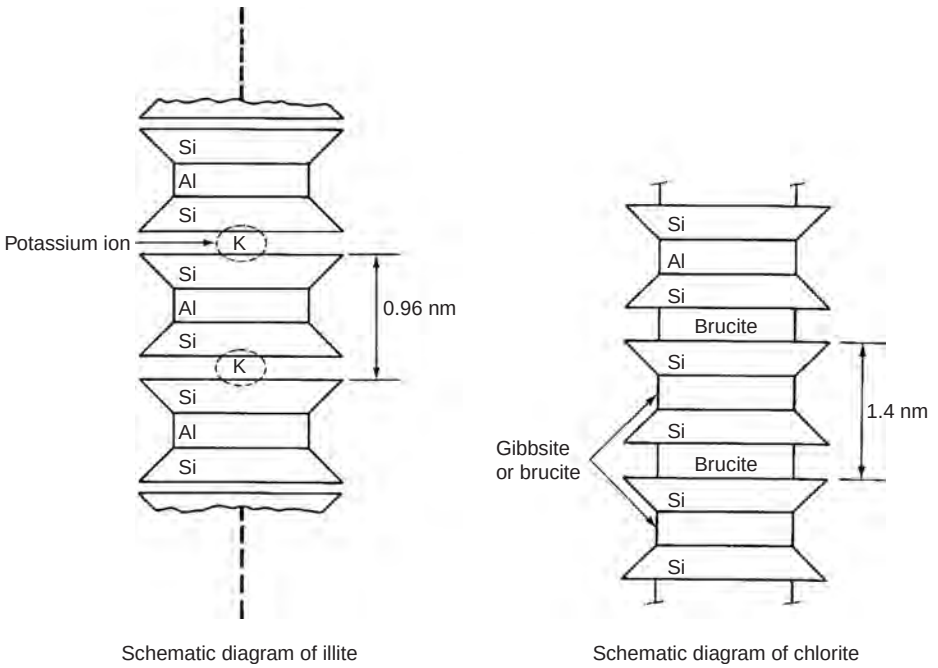


Figure 7.5 Schematic diagram of the structure of illite and chlorite molecule (Holtz and Kovacs, 1981).

the fluctuating dipole causes a weak bond known as Van der Waal's bond. These bonds are additive between atoms, and although weak compared to hydrogen bonds, they decrease less with distance from the nucleus than the latter.

7.2.3 Bonding between layers of clay minerals

Bonding between layers of clay minerals occurs in five ways:

- *Van der Waal's forces* – these occur in most clay minerals,
- *hydrogen bonding*, e.g. between the positive end of the OH at the base of the Al octohedra and the negative end of the O at the top of the silica tetrahedra in kaolinite (Figure 7.3),
- *hydrogen bonding with polar water* – as shown in Figure 7.4,
- *exchangeable cation bonding* – cations, e.g. Ca^{++}, Na^{+} act to bond between the negatively charged surfaces of such clay minerals as montmorillonite. The negative charge is brought about by substitution of cations within the sheet structure, e.g. Mg^{++} for Al^{+++}. This is known as isomorphous substitution and is permanent. The interlayer cations are not permanent and may be substituted by other cations,
- *inter-layer cation bonding* – cations such as K^{+}, Mg^{++}, Fe^{++} which fit in the space between layers giving a strong bond. Examples are micas and chlorites. These cations are not affected by the presence of water. Figure 7.5 shows the structure of illite and chlorite.

Table 7.1 Schematic structures of the common clay minerals (Adapted from Lambe and Whitman 1981).

Mineral	Structure symbol	Mineral	Structure symbol
Serpentine		Muscovite	
Kaolinite		Vermiculite	
Halloysite ($4H_2O$)		Illite	
Halloysite ($2H_2O$)		Montmorillonite	
Talc		Nontronite	
Pyrophyllite		Chlorite	

Table 7.1 lists the predominant bonding for the common clay minerals.

7.3 INTERACTION BETWEEN WATER AND CLAY MINERALS

7.3.1 Adsorbed water

There is much evidence that water is attracted to clay minerals, e.g.:

- Dry soils take up water from the atmosphere, even at low relative humidity,
- Soils wet up and swell when given access to water,
- Temperatures above 100°C are required to drive off all water from soils.

Much of the water in soil is adsorbed, i.e. the electrical charge of the water dipoles and the electrical charge associated with the surface of the clay minerals attract each other.

There are several explanations for this as shown in Figure 7.6:

- *hydrogen bonding*, where the positive H side of the water dipole is attracted to surface oxygen, or the negative O side of the water dipole is attracted to surface hydroxyls.

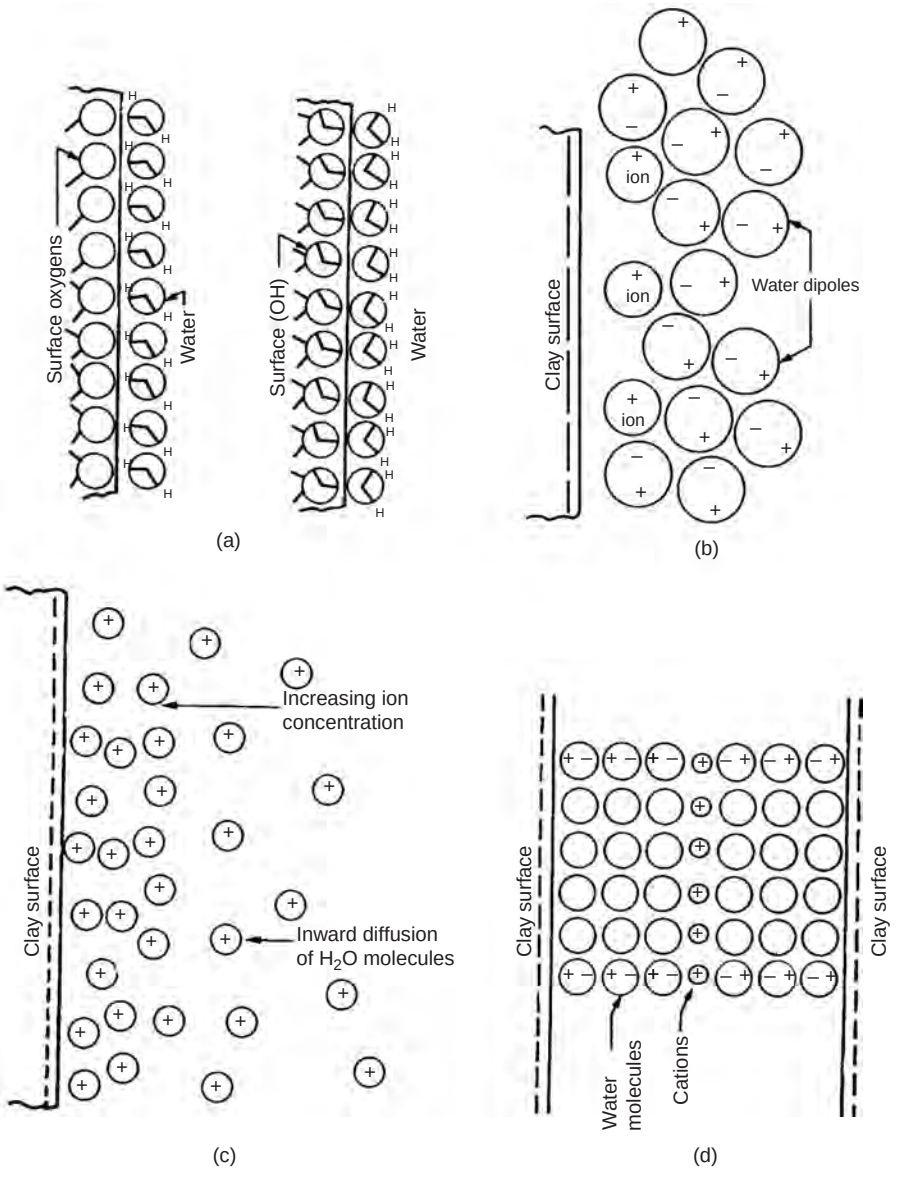


Figure 7.6 Possible mechanisms of water adsorption by clay surfaces: (a) hydrogen bonding; (b) ion hydration; (c) attraction by osmosis; (d) dipole attraction (Mitchell 1976).

- *ion hydration*, where water dipoles are attracted with cations, which are in turn attracted by the negative charge of the surface of the clay mineral.
- *osmosis*: as discussed below, there is a greater concentration of cations close to the surface of the clay than further away. Water molecules tend to diffuse towards the surface in an attempt to equalise concentrations.

Edge view	Typical thickness (nm)	Typical diameter (nm)	Specific surface (km^2/kg)
Montmorillonite	3	100–1000	0.8
Illite	30	10 000	0.08
Chlorite	30	10 000	0.08
Kaolinite	50–2000	300–4000	0.015

Figure 7.7 Average values of the relative sizes, thicknesses and specific surfaces of the common clay minerals (Holtz and Kovacs, 1981).

– *dipole attraction* – water dipoles orienting themselves as shown in Figure 7.6(d), with a central cation to equalise charges.

Some clay minerals, particularly those of the smectite, or montmorillonite group, have undergone significant isomorphous substitution giving a large net negative charge on the surface. They also tend to have particles which are very thin giving a large exposed surface area. As a result, these clays tend to attract water more than those which do not have a large negative surface charge and are thicker.

The surface area of clay minerals is measured by the specific surface, where

$$\text{specific surface} = \frac{\text{surface area}}{\text{volume}} \tag{7.1}$$

$$\text{or} = \frac{\text{surface area}}{\text{mass}} \tag{7.2}$$

Figure 7.7 shows the typical dimensions and specific surface for four common clay minerals.

7.3.2 Cation exchange

As outlined above, one of the bonding mechanisms present in the montmorillonite (or smectite) group of clay minerals is exchangeable cation bonding. The cations, which are held in the space between sheets of clay minerals, are able to be exchanged for other cations and, in the process, the properties of the clay can be altered. This process is known as cation exchange.

This source of cation exchange is the most important one in clay minerals except for kaolinite. Other sources are:

- Broken bonds: electrostatic charges on the edges of particles and particle surfaces. This is the major cause of exchange in kaolinite.
- Replacement of the hydrogen of an exposed hydroxyl.

The most commonly present cations are Ca^{++}, Mg^{++}, Na^{+} and K^{+}. The ease with which the cations replace each other is dependent on:

- Valence – divalent ($^{++}$) substitute readily for univalent ($^{+}$), e.g. Ca^{++} for Na^{+}
- Relative abundance or concentration of the ion type,
- Ion size, or charge density – smaller high charge cations will displace large, low charge cations.

A typical replacement series is:

$$Na^{+} < Li^{+} < K^{+} < Mg^{++} < Ca^{++} < Al^{+++} < Fe^{+++}$$

However, Ca^{++} can be replaced by Na^{+} if the concentration of Na^{+} is sufficiently high. The rate at which cation exchange occurs depends on:

- Clay type – for example, exchange will be almost instantaneous in kaolinite, a few hours in illite, and longer in montmorillonite,
- Concentration of cations in the soil water,
- Temperature.

The quantity of exchangeable cations required to balance the charge deficiency of a clay is known as the cation exchange capacity (CEC) and is usually expressed as milliequivalents per 100 grams of dry clay.

7.3.3 Formation of diffuse double layer

In a dry soil, the exchangeable cations are held tightly to the negatively charged clay surface. Excess cations and their associated anions precipitate as salts.

When placed in water, the salts go into solution and because of the high concentration of adsorbed cations near the clay surface these diffuse from the clay surface as they repel each other. However, this is counteracted by the negative charge on the clay surface. The anions in solution are repelled by the negative charge surface of the clay (but attracted to the cations). The overall effect is to result in a distribution of ions shown in Figure 7.8.

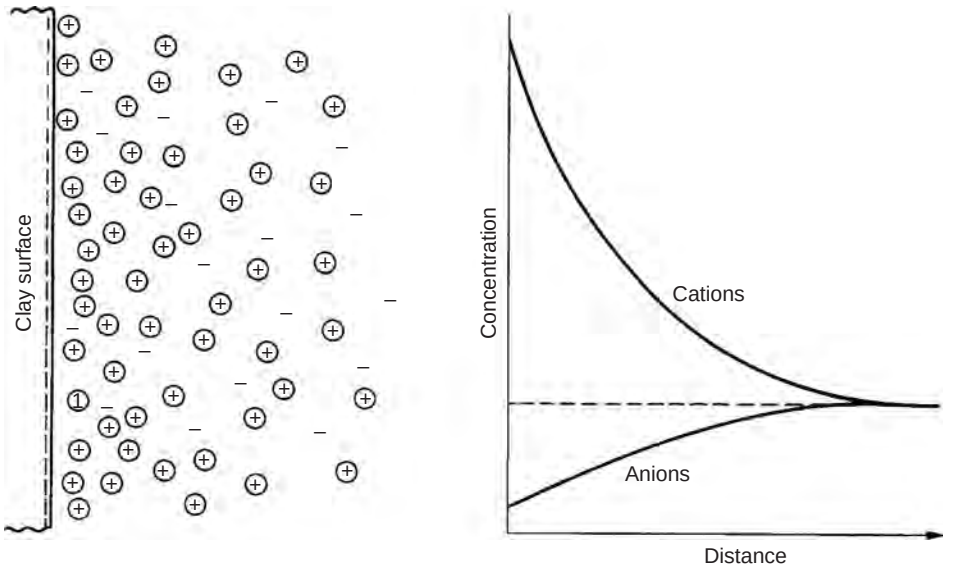


Figure 7.8 Distribution of ions adjacent to a clay surface according to the concept of the diffuse double layer (Mitchell, 1976).

The negative surface and the distributed ions adjacent are together known as the diffuse double layer. Details are discussed in Mitchell (1976, 1993).

The resulting electrical potential varies with distance from the clay surface as shown in Figure 7.9. The potential is negative, reflecting the large negative charge on the clay surface.

7.3.4 Mechanism of dispersion

When two clay particles come near each other, the potential fields overlap, leading to repulsion if the particles are close enough. These repulsive forces are counteracted by Van der Waal's attractive forces as shown in Figure 7.10. If the repulsive forces are greater than the Van der Waal's forces the soil will disperse. In cases where the repulsive forces are small, the Van der Waals attractive forces dominate and flocculation results.

The repulsive forces in the diffuse double layer are affected by several factors:

a) *Electrolyte concentration*: As shown in Figure 7.11, a high concentration of dissolved salt in the soil water leads to a smaller diffuse double layer (as the greater concentration of cations (Na^+) more readily overcomes the negative charge on the clay surface). Hence the repulsive forces are lower.
b) *Cation valence*: Exchange of Na^+ cations by Ca^{++} cations leads to a smaller, higher charge density diffuse double layer and hence lower repulsive forces.

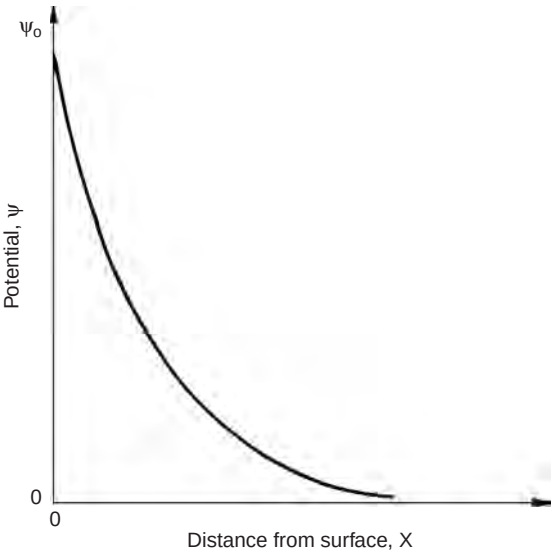


Figure 7.9 Variation in electrical potential with distance from the clay surface (Adapted from Mitchell, 1976).

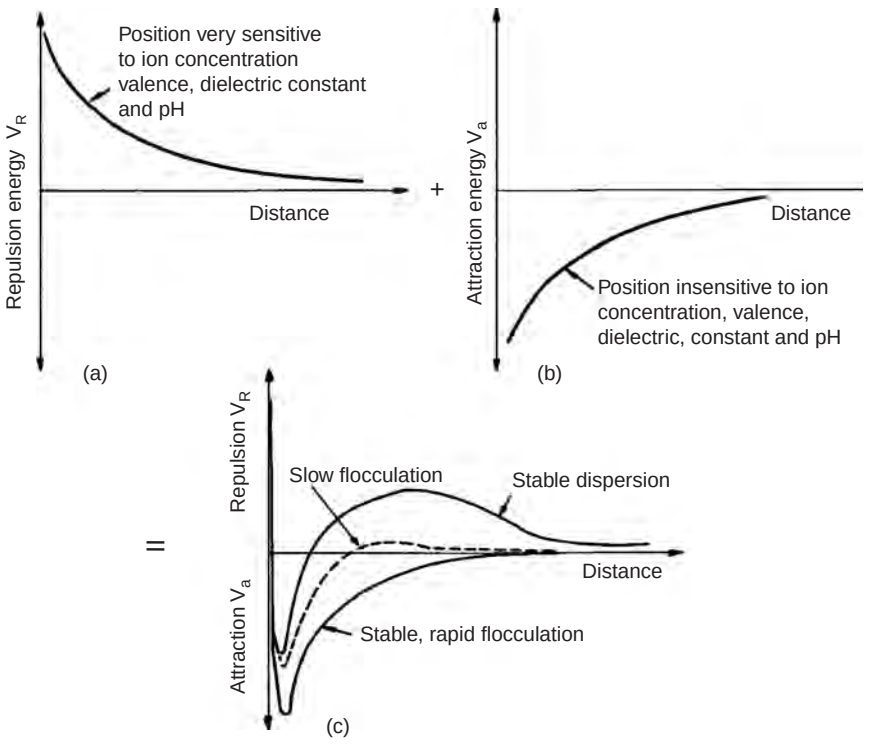


Figure 7.10 Interaction of (a) repulsive and (b) Van der Waals attractive forces to give (c) curves of net energy of repulsion or attraction (Adapted from Mitchell, 1976).

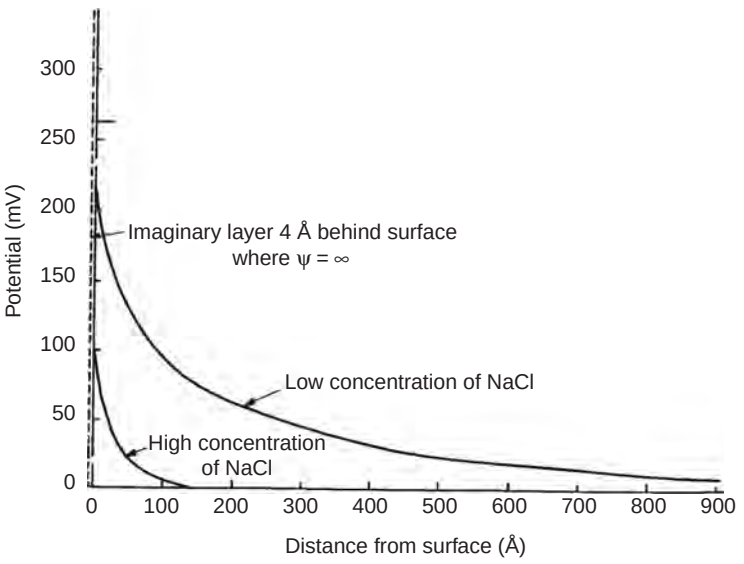


Figure 7.11 Effect of electrolyte concentration on diffuse double layer potential for montmorillonite (Adapted from Mitchell, 1976).

Other factors which affect the diffuse double layer include:

- dielectric constant of the electrolyte
- temperature

More details are given in Mitchell (1976, 1993).

7.4 IDENTIFICATION OF CLAY MINERALS

When required, the identification of clay mineral(s) present in a soil is usually carried out using at least two of the following techniques. It should be noted that most soils have clay sized particles mixed with silt and sand sized particles, the latter usually being quartz or other rock mineral. The results of the clay mineralogy must therefore be considered allowing for that proportion of the soil they represent in total.

7.4.1 X-ray diffraction

A sample of the powdered soil is subject to x-rays, which are diffracted depending on the mineral crystals present. Each clay mineral has a characteristic pattern which is known. Mitchell (1976, 1993) discusses the technique.

7.4.2 Differential Thermal Analysis (DTA)

A sample of the soil and an inert substance are heated at a controlled rate (10°C/minute) up to 1000°C, and the differences in temperature in the soil and the inert substance are recorded.

Endothermic reactions (i.e. heat being absorbed) are usually related to the removal of:

- Adsorbed water
- Water of hydration

Exothermic reactions are usually related to re-crystallisation and oxidation. Each clay mineral has a characteristic thermogram allowing its identification. Mitchell (1976, 1993) indicates that the method can be calibrated for quantitative analysis, accurate to the determined value plus or minus 5%.

7.4.3 Electron microscopy

Scanning electron microscopy can be used to assist in identification of clay minerals. Magnifications from X20 to X150,000 can be used and the micrographs compared with those for known clay minerals. The main clay minerals have distinctly different micrographs.

In practice the presence of several clay minerals, and silt and sand complicates identification but, coupled with x-ray diffraction or differential thermal analysis, the method can be useful.

The above methods tend to be qualitative rather than quantitative. Mitchell (1976) indicates that a quantitative analysis can be made using DTA, glycol adsorption, cation exchange capacity and K_2O content. A semi quantitative analysis can be done by examining the silt and sand fractions under a microscope to ascertain the proportion of non clay minerals. The clay size % (finer than 0.002 mm) can be obtained by grain size analysis and the proportion of the clay minerals in the clay size obtained from x-ray diffraction. The authors' experience is that few organisations are willing to quantify the amount of different clay minerals present.

Apart from the above methods a rough idea of which clay minerals are present can be obtained from the Atterberg limits.

7.4.4 Atterberg limits

As shown in Figure 7.12 the position of the soil on the Casagrande plasticity chart can give an indication of which minerals are present. It should be remembered that most soils contain several clay minerals, so the Atterberg limits may not fall exactly in the shaded zones.

7.4.5 The activity of the soil

$$\text{Activity} = \frac{\text{plasticity index}}{\text{clay fraction}} \tag{7.3}$$

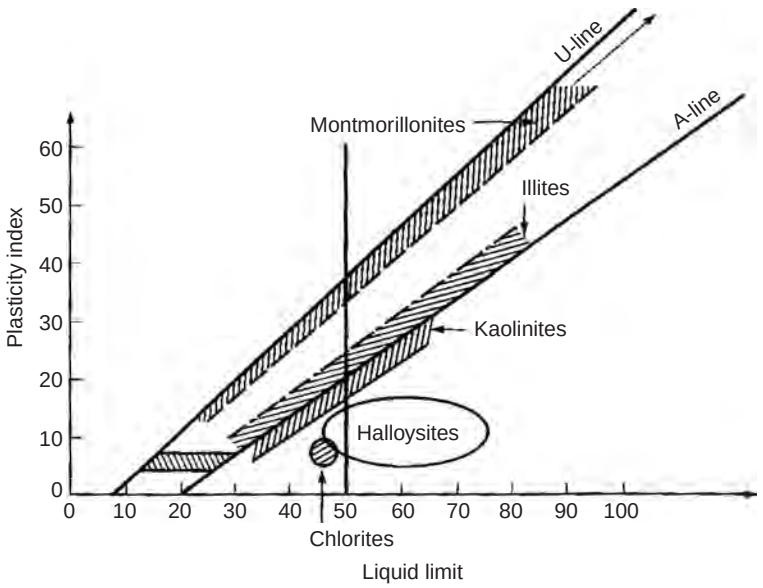


Figure 7.12 Location of common clay minerals on Casagrande plasticity chart (Holtz and Kovacs, 1981).

Table 7.2 Activities of various minerals (Holtz and Kovacs, 1981).

Mineral	*Activity*
Na-montmorillonite	4–7
Ca-montmorillonite	1.5
Illite	0.5–1.3
Kaolinite	0.3–0.5
Halloysite (dehydrated)	0.5
Halloysite (hydrated)	0.1
Attapulgite	0.5–1.2
Allophane	0.5–1.2
Mica (muscovite)	0.2
Calcite	0.2
Quartz	0

Note: Activity = (plasticity index)/(clay fraction)

where clay fraction = % finer than 0.002 mm. Table 7.2 shows typical values. Soils with high activity are likely to contain montmorillonite and illite, while those with lower activities are likely to contain kaolinite. However the plasticity index is carried out on that part of the soil which includes clay, silt and fine sand, so its value relates also to the amount of clay present.

7.5 ENGINEERING PROPERTIES OF CLAY SOILS RELATED TO THE TYPES OF CLAY MINERALS

The engineering properties of clay soils depend on compositional factors (Mitchell 1976, 1993):

- type of clay minerals present
- amount of each mineral
- type of adsorbed cations (and anions)
- organic content
- shape and size distribution of particles
- pore water composition

And on environmental factors:

- water content
- density
- confining pressure
- fabric
- availability of water
- temperature

Virtually all clay soils in nature are mixtures of clay and silt size particles (and sometimes sand), not just clay size particles. The silt and sand size particles are usually rounded or sub-rounded and are derived from the parent rock. The most abundant mineral present is usually quartz, followed by feldspar and mica.

7.5.1 Dispersivity

Soils in which the clay particles will detach from each other and from the soil structure without a flow of water, and go into suspension are termed dispersive clays.

The dispersivity of a soil is directly related to its clay mineralogy. In particular soils with a high exchangeable sodium percentage such as montmorillonite present, tend to be dispersive, while kaolinite and related minerals (halloysite) are non-dispersive. Soils with illite present tend to be moderately dispersive.

The dispersivity depends also on the pore water chemistry since, as discussed above this affects the diffuse double layer geometry and electrical charge. In particular low electrolyte (pore water) salt concentrations lead to a large diffuse double layer and greater dispersivity. Hence percolation of a saline potentially dispersive soil with fresh water can result in the soil changing from non-dispersive to dispersive.

Cation exchange of say Ca^{++} for Na^{+} leads to a smaller diffuse double layer, and lower dispersivity. Hence addition of lime (CaO or $Ca(OH)_2$), or gypsum ($CaSO_4$) leads to cation exchange and reduced dispersivity.

There have been many failures of dams constructed of dispersive clays. Mitchell (1993) indicates these have been mostly low to medium plasticity (CL and CL-CH) clays that contained montmorillonite. Bell and Maud (1994) concur with this view. The authors experience is similar, that is the most susceptible soils are not those with

Table 7.3 Swelling index values for several minerals (Olson and Mesri, 1970).

Mineral	*Pore fluid, adsorbed cations electrolyte concentration, in gram equivalent weights per litre*	*Void ratio at effective consolidation pressure of 100 psf*	*Swelling index*
Kaolinite	Water, sodium, 1	0.95	0.08
	Water, sodium, 1×10^{-4}	1.05	0.08
	Water, calcium, 1	0.94	0.07
	Water, calcium, 1×10^{-4}	0.98	0.07
	Ethyl alcohol	1.10	0.06
	Carbon tetrachloride	1.10	0.05
	Dry air	1.36	0.04
Illite	Water, sodium, 1	1.77	0.37
	Water, sodium, 1×10^{-3}	2.50	0.65
	Water, calcium, 1	1.51	0.28
	Water, calcium, 1×10^{-3}	1.59	0.31
	Ethyl alcohol	1.48	0.19
	Carbon tetrachloride	1.14	0.04
	Dry air	1.46	0.04
Smectite	Water, sodium, 1×10^{-1}	5.40	1.53
(montmorillonite)	Water, sodium, 5×10^{-4}	11.15	3.60
	Water, calcium, 1	1.84	0.26
	Water, calcium, 1×10^{-3}	2.18	0.34
	Ethyl alcohol	1.49	0.10
	Carbon tetrachloride	1.21	0.03
Muscovite	Water	2.19	0.42
	Carbon tetrachloride	1.98	0.35
	Dry air	2.29	0.41
Sand			0.01–0.03

a high plasticity, but those with limited clay size fractions sufficient only to give low to medium plasticity.

Sherard *et al.* (1976a, b) however tested some soils as dispersive in the pinhole test with % passing 0.005 mm greater than 50%. The better performance of the higher plasticity clays probably relates to greater resistance to erosion and greater likelihood for cracks to close as the soil swells.

Sherard *et al.* (1976a, b) indicate that based on their tests, soils with less than 10% finer than 0.005 mm may not have enough clay to support dispersive piping.

7.5.2 Shrink and swell characteristics

The shrink and swell characteristics of a soil can be related to clay mineralogy. Table 7.3 shows swelling index ($de/d(\log p')$), where e = void ratio, p' = effective stress on the soil) values for several clay minerals, with varying electrolyte concentrations and for different cations in the electrolyte.

It can be seen that:

– At the same pore fluid adsorbed cations concentration, montmorillonite swells to a greater void ratio and has a higher swelling index than illite, which in turn swells more than kaolinite.

– Exchange of Na^+ by Ca^{++} in montmorillonite (smectite) significantly reduces swell potential, but has only a minor effect on kaolinite.
– Increase of concentration of Na^+ causes a marked decrease in the swelling index of montmorillonite, but no change in the swelling index of kaolinite.

These phenomena are to be expected given the large surface area and higher negative charge on the surface of montmorillonite when compared to illite and kaolinite.

7.5.3 Shear strength

The shear strength, parameters c′, ϕ' (peak strength) and C'_R, ϕ'_R (residual strength) are dependent on the clay fraction percentage as shown in Figures 6.4, 6.5 and 6.6. Clays with a high clay fraction percentage exhibit lower shear strengths and a greater reduction from peak to residual strength. As shown in Figures 6.5 and 6.6 and Figure 7.13 the shear strength is also dependent on the types of clay minerals present, with lowest strength for pure montmorillonites, highest for kaolinite. However for normal soils, the behaviour is also influenced by the mixture of clay minerals present, and the presence of silt and clay sized particles.

7.5.4 Erosion properties

As discussed in Section 8.3.4.6 the dispersivity of the soil affects the critical hydraulic shear stress at which erosion initiates but has relatively little effect on the rate of erosion. Emerson Class number 1 and 2, and Sherard Pinhole dispersion D1 and D2 soils have critical shear stresses less than 5 Pa so erosion will initiate even in narrow cracks at low gradients.

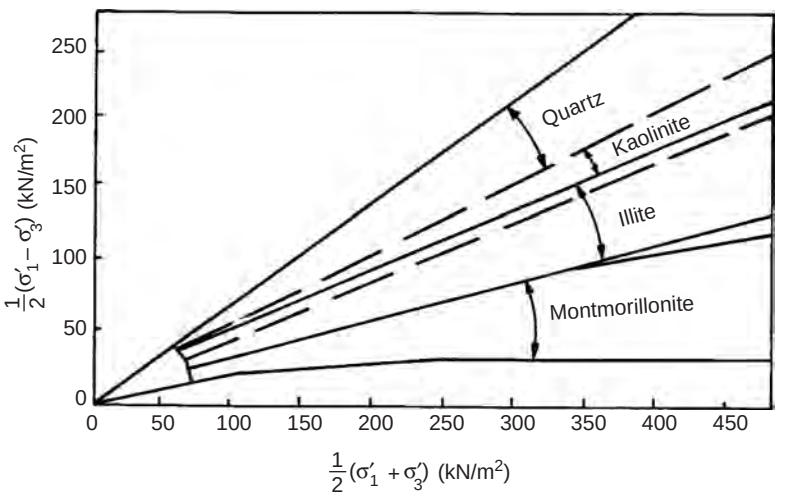


Figure 7.13 Ranges in effective stress failure envelopes for pure clay minerals and quartz (Olson 1974).

7.6 IDENTIFICATION OF DISPERSIVE SOILS

7.6.1 Laboratory tests

There are several laboratory tests which can be used to determine the dispersivity of a soil.

7.6.1.1 *Emerson class number*

In this method, the soil is sieved through a 4.75 mm sieve and collected on a 2.36 mm sieve and tested as outlined in Figure 7.14. The test procedure is detailed in Standards Australia (1997), test AS 1289, 3.81 and USBR (1979).

The test is carried out in distilled water, but may be repeated in water from the dam, or groundwater. This often gives significantly different results due to the presence of dissolved salts in the water (higher salt content gives less dispersive results).

However caution should be applied on relying on the tests in reservoir water because under flood conditions the dissolved salt content may drop in the reservoir so the tests in distilled water may be more applicable.

The soils are graded according to class, with Class 1 being the highly dispersive, Class 8 non dispersive. Soils with Emerson Class 1 to 4 but in particular class 1 and 2 need to be treated with extra caution in dam construction.

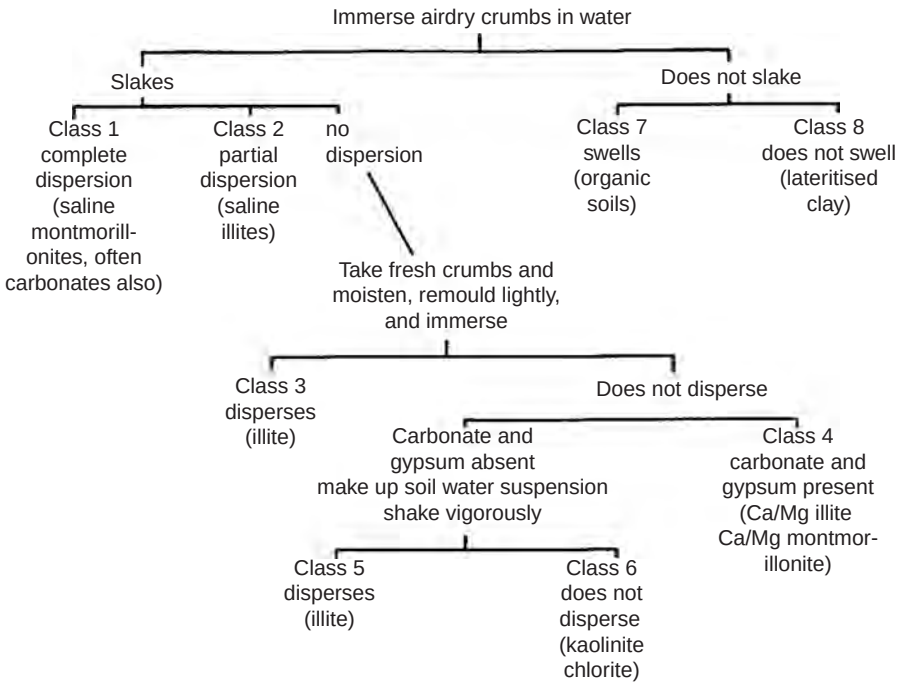

Immerse airdry crumbs in water
Slakes
Does not slake
Class 1 complete dispersion (saline montmorill-onites, often carbonates also)
Class 2 partial dispersion (saline illites)
no dispersion
Class 7 swells (organic soils)
Class 8 does not swell (lateritised clay)
Take fresh crumbs and moisten, remould lightly, and immerse
Class 3 disperses (illite)
Does not disperse
Carbonate and gypsum absent make up soil water suspension shake vigorously
Class 4 carbonate and gypsum present (Ca/Mg illite Ca/Mg montmor-illonite)
Class 5 disperses (illite)
Class 6 does not disperse (kaolinite chlorite)

Figure 7.14 Determination of the Emerson class number of a soil (Ingles & Metcalf, 1972).

Sherard *et al.* (1976b) indicate that, if a soil tests as dispersive in the crumb test, it will also test as dispersive in the pinhole test, but 40% of soils testing as dispersive in the pinhole test, test as non dispersive in the crumb test (note that Sherard *et al.*, 1976b, use different classes to the Australian standard). This is presumably due to these soils being erodible as measured by the pinhole test but not chemically dispersive.

The Emerson test allows identification of dispersive soils, but does not provide a measure of their erodibility. It is inexpensive, and a useful first check on dispersivity.

7.6.1.2 *Soil Conservation Service test*

Soil Conservation Service test, also known as the double hydrometer test, or percent dispersion test (Standards Australia, 1997, test AS1289, 3.8.2 and ASTM 2001 test D42291-99).

This involves two hydrometer tests on soil sieved through a 2.36 mm sieve. The hydrometer tests are carried out with dispersant and without. The percent dispersion is:

$$\frac{P}{Q} \times 100 \tag{7.4}$$

where P = percentage of soil finer than 0.005 mm for the test without dispersant and Q = percentage of soil finer than 0.005 mm for the test with dispersant.

Sherard *et al.* (1976a) indicate that soils with a percent dispersion greater than 50% are susceptible to dispersion and piping failure in dams and those with a percent dispersion less than 15% are not susceptible. A percent dispersion of less than 30% is unlikely to test dispersive in the pinhole test. Bell and Maud (1994) indicate that soils with >50% dispersion are regarded as highly dispersive, 30–50% moderately dispersive, 30% to 15% slightly dispersive and <15%, non dispersive.

7.6.1.3 *Pinhole dispersion classification*

Pinhole dispersion classification, also known as the pinhole test, or Sherard pinhole test (Standards Australia 1997, test AS1289, 3.8.3 and ASTM 1998, test D4647-93).

This test was developed by Sherard *et al.* (1976b). A 1.0 mm diameter hole is preformed in soil to be tested, and water passed through the hole under varying heads and for varying durations. The soil is sieved through a 2.36 mm sieve and compacted at approximately the plastic limit to a density ratio of 95% (to simulate conditions in a dam embankment with a crack or hole in the soil).

Soils which tested as D1 and D2 were found by Sherard *et al.* (1976a) to have suffered piping failure in earth dams and severe erosion damage by rainfall in embankments and natural deposits while those with ND1 and ND2 classification had not.

As for the Emerson class number, the results are dependent on the chemistry of the water used for the test (the standard test uses distilled water). The method is relatively simple with moderate cost, and has the advantage that "it identifies soil erodibility directly, rather than indirectly". All dispersive soils are erodible to some degree – usually highly – but erodibility depends on other factors than the clay fraction. Craft and Acciardi (1984) describe some modifications to the Sherard *et al.* (1976b) method. These are incorporated in Standards Australia (1997) and ASTM (1998).

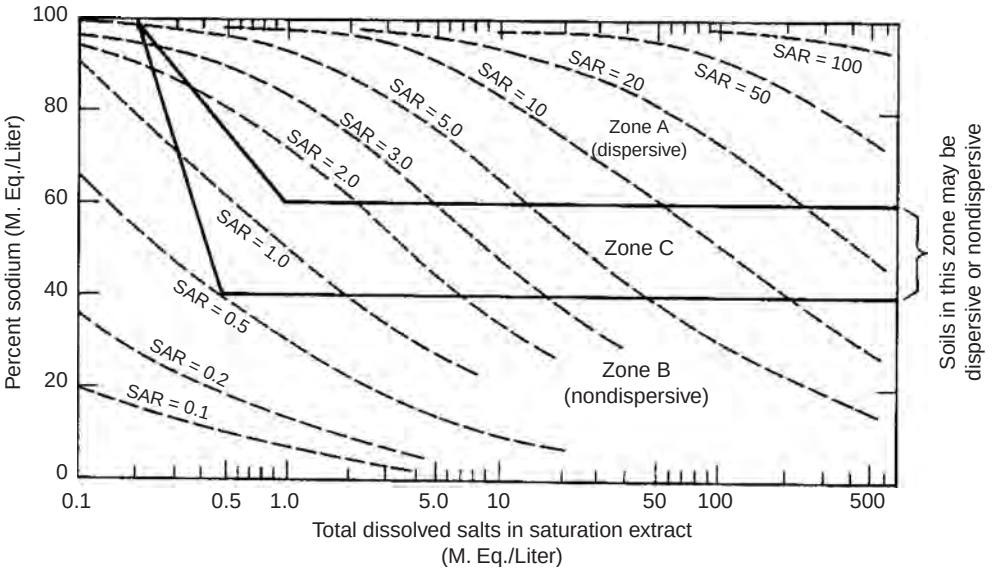


Figure 7.15 Relationship between dispersivity and dissolved pore water salts content (Sherard *et al.*, 1976a).

Gerber and Harmse (1987) found that some dispersive soils with free salts in solution in the pore water were not identified as dispersive by the pinhole, SCS or crumb tests.

7.6.1.4 *Chemical tests*

Based on correlation with many dam failures and soil from dams which have leaked continuously (without any filters to control erosion) and not failed (Sherard *et al.*, 1976a) proposed Figure 7.15 to determine the dispersivity of soil.

In this figure

$$\text{percent sodium} = \frac{Na^+}{\text{Total dissolved salts}} \times 100$$

$$\text{or} = \frac{Na^+}{Ca^{++} + Mg^{++} + Na^+ + K^+} \times 100$$

$$\text{Sodium adsorption ratio (SAR)} = \frac{Na^+}{\left[\frac{1}{2}(Ca^{++} + Mg^{++})\right]^{1/2}}$$

in which Na^+, Ca^{++}, and Mg^{++} are measured in milliequivalents per litre of saturation extract.

Sherard *et al.* (1976a) indicate that the case histories and pinhole tests confirm that the main factor governing dispersivity is the relative content of pore-water sodium, and claimed a general relationship, as shown in Figure 7.15, valid for most soils, has now been established with considerable confidence. The zones given in Figure 7.15 represent:

Zone A – Much experience shows that damaged and failed dams all over the world have been constructed of these dispersive soils. Almost all soils are dispersive in the pinhole test.

Zone B – the great majority of these soils are non-dispersive. These are the soils generally considered "ordinary erosion resistant clays", but include silts of low plasticity (ML), also non-dispersive. A small percentage of exceptional soils in Zone B erode in the pinhole test in exactly the same fashion as soils of Zone A, and some of these can be identified only by the pinhole test.

Zone C – soils in this group may range from dispersive to non-dispersive. This group contains a few soils which give intermediate reaction in pinhole tests, with apparently colloidal erosion but at a very slow rate compared with soils of Zone A.

These relationships are only valid for relatively pure eroding waters (such as TDS less than 0.5 meq/1 or about 300 ppm). This is because they are based on results of pinhole tests using distilled water and case histories of erosion in nature caused mainly by rainwater and relatively pure reservoir waters.

While Sherard *et al.* (1976a) found good agreement with this test and field behaviour, Bell and Maud (1994) indicate the method was unreliable in South African conditions. They quote Craft and Acciardi (1984) as concluding that the extraction of the soil water is difficult because of the low volumes used. They indicate it may be used where prior testing correlates well with the pinhole test.

Sherard *et al.* (1976a) did have small number of soils which tested as erodible in the pinhole test falling in Zone B. These soils were not from failed dams or areas showing erosion.

Other terms which are useful for assessing the likely dispersivity of a soil are:

$$\text{Exchangeable sodium percentage (ESP)} = \frac{\text{exchangeable sodium}}{\text{cation exchange capacity}} \times 100$$

$$\text{or} = \frac{Na^+}{Na^+ + K^+ + Ca^{++} + Mg^{++}} \times 100$$

where the units are milliequivalents/100 grams of dry soil.

ICOLD (1990) indicate that soils with ESP of 10% or above, which are subject to having free salts leached by seepage of relatively pure water are classified as dispersive. They indicate that Australian research showed that soils are dispersive if SAR exceeded 2.

ESP and SAR are determined by chemical analysis of the soil pore water extracted by vacuum from saturated samples.

Ingles and Metcalf (1972) suggest that to assess dispersivity potential, the exchangeable sodium percentage and total ionic concentration of the water, e.g. the

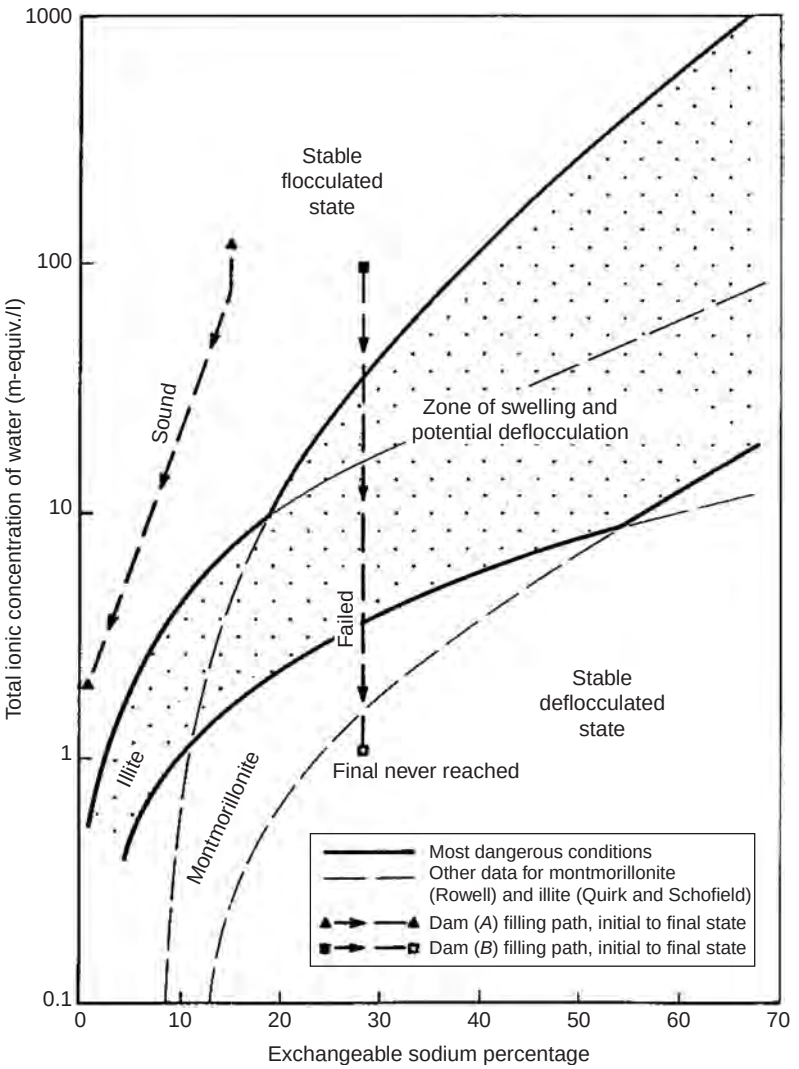


Figure 7.16 Estimation of dispersivity from exchangeable sodium percentage of the pore water, and total ionic concentration of the seepage water (Ingles and Metcalf 1972).

water to be stored in the dam, should be determined. Then for montmorillonite and illite clays, Figure 7.16 is used to determine whether the soil is in a flocculated, or dispersed (deflocculated) state, and whether a change in pore water chemistry during filling of the reservoir can lead to dispersion.

In Figure 7.16 filling of the storage for Dam A maintains a stable flocculated state, but for Dam B, dispersion would occur. This can be readily related to the diffuse double layer concept, where a change in concentration of salts in the soil water can lead to a larger diffuse double layer, and a tendency to disperse (Dam B), but exchange of sodium

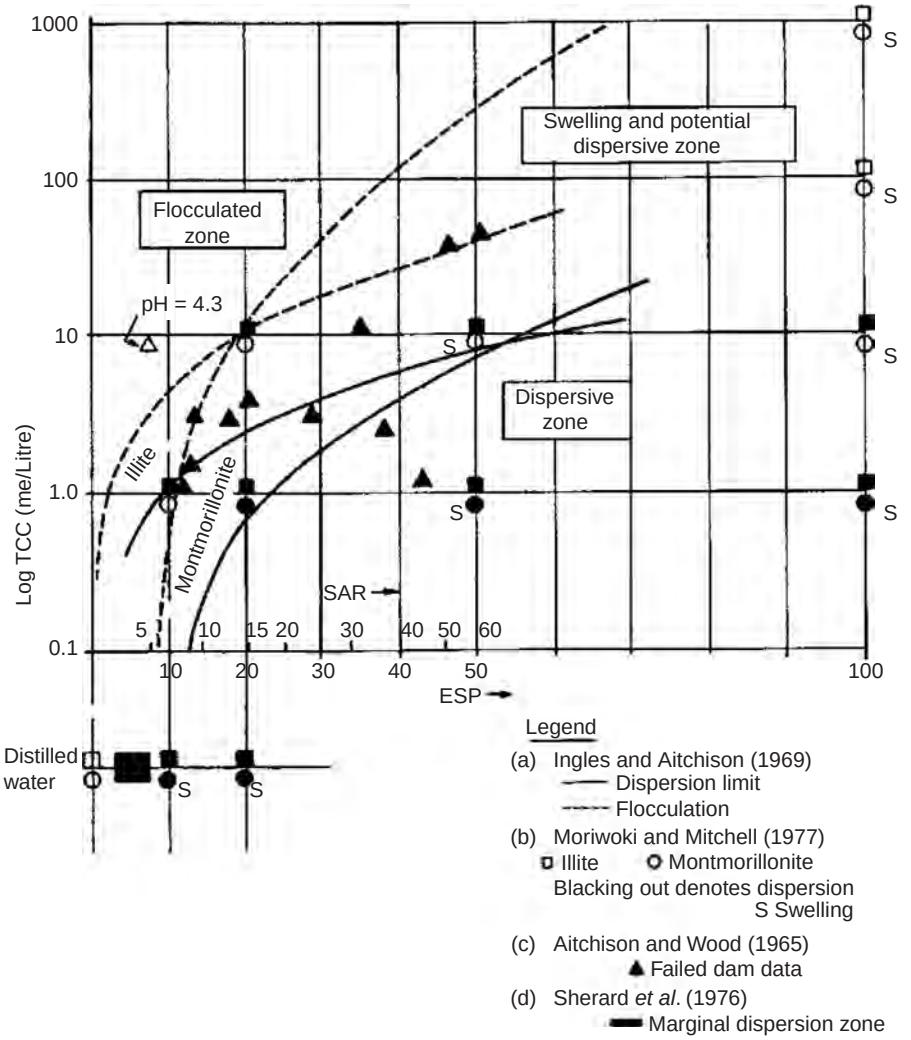


Figure 7.17 Estimation of dispersivity from exchangeable sodium percentage (McDonald, Stone and Ingles, 1981).

cations by say calcium would retain a stable flocculated structure even though the total concentration was reduced (Dam A). McDonald *et al.* (1981) produced Figure 7.17 to include some work by Moriwaki and Mitchell (1977).

This indicated that the Ingles and Metcalf (1972) boundaries were reasonable except that dispersion could occur in montmorillonites at higher ESP than indicated by the earlier work. McDonald *et al.* (1981) suggested the use of the pinhole test for situations where water would be flowing (e.g. a crack in a dam) and Emerson test for quiescent conditions, e.g. reservoir. They also point out that the ESP can vary a lot for soils in any dam and make use of cheaper tests such as Emerson and

pinhole tests more attractive. It is also pointed out that where soils which are rich in bicarbonates are allowed to dry out, precipitation of relatively insoluble $CaCO_3$ may occur, resulting in a lower Ca^{++} ion concentration and hence an increased potential for dispersion.

It should be noted that Sherard *et al.* (1976a) believe that such transformation of a stable, flocculated soil into a dispersive soil by the seepage water is unlikely, citing that most failures occur on first filling, pinhole tests eroding at the start of a test, or not at all (even after long periods of flow), and that there is no reason why the ESP should reduce (their experience being the contrary).

7.6.1.5 *Recommended approach*

Most authors consider that it is necessary to use more than one test to ascertain the dispersivity of a soil. Sherard and Decker (1977) suggest that four tests should be used: Soil Conservation Service, pinhole, Emerson and chemical test. They were of the opinion that the pinhole test was best. Similar results were obtained using the pinhole and Emerson tests.

- The pinhole test was not particularly sensitive to compaction water content.
- Only approximate agreement was found between the SCS test and the pinhole and Emerson tests.
- The Sherard *et al.* (1976a) chemical test shown in Figure 7.15 showed a soil to be dispersive when it was known not to be (by its field behaviour and other tests).

Bell and Maud (1994) suggested using the crumb test, SCS dispersion test, ESP/CED, SAR and PH, weighting the results, taking most account of the ESP/CEC, followed by the crumb and SCS dispersion test.

The authors have used the Emerson class number and pinhole tests: usually in a ratio of 2 or 3 crumb tests, to 1 pinhole test, reflecting the greater cost of the pinhole test. If one accepts that the objective is only to identify potential problems this would seem an adequate approach, but better supplemented by some SCS dispersion tests or chemical tests. The chemical approach in Figure 7.17 has attractions in that it allows a prediction of behaviour with altered water quality and can be related to clay mineralogy. However, the Emerson class and pinhole tests can also be related to changes in water quality by carrying out these tests in water other than distilled water.

It should be recognised that the tests identify the degree of dispersivity of soils. They do not measure quantitatively the rate of erosion, or critical hydraulic shear stress at which erosion begins. Wan and Fell (2002, 2004a) have developed the Hole Erosion Test and Slot Erosion Test which allow this quantification.

7.6.2 Field identification and other factors

While laboratory tests are a useful way of identifying dispersive soils, much can be determined by observing the behaviour of the soils in the field, e.g.:

- The presence of deep erosion gullies and piping failure in existing small farm dams usually indicates the presence of dispersive soils – see Figures 7.18 and 7.1.



Figure 7.18 Deep erosion gullying in dispersive soils (Soil Conservation Service of NSW).

- Erosion of road cuttings, tunnel erosion along gully lines and erosion of weathered or clay infilled rock joints may indicate potentially dispersive soils.
- The presence of cloudy water in farm dams and puddles of water after rain indicates dispersive soils.
- The presence of numerous sinkholes; e.g. on the surface of the dam embankment. These form as rain water enters desiccation cracks and erodes the soil on the sides of the cracks.

One can infer the clay mineralogy from such observational techniques. Some guidelines are given in Tables 7.4 and 7.5 reproduced from Ingles and Metcalf (1972).

The geology of the area can also be a guide to dispersivity.

Sherard and Decker (1977) indicate that:

- Many dispersive clays are of alluvial origin (The authors' experience is similar but there are many non-dispersive alluvial clays. Some slopewash clays are also dispersive).
- Some soils derived from shales and claystones laid down in a marine environment are also dispersive.
- Soils derived from weathering of igneous and metamorphic rocks are almost all non dispersive (but may be erodible, e.g. silty sand derived from grandiorite).
- Soils with a high organic content are unlikely to be dispersive (this needs to be treated with caution, since many "black cotton" soils are dispersive.

Table 7.4 Clay mineral identification from the environment (Ingles and Metcalf, 1972).

Observation	*Dominant clay component*[(1)]
Turbid water of strong yellow-brown to red-brown colour	Montmorillonites, illite plus soil salinity
Clear waters	Calcium, magnesium or ironrich soil, highly acid soil sands
Clear waters with a bluish cast	Non-saline kaolins
Erosion gullies and/or field tunnelling in the natural soil	Saline clays, usually montmorillonites
As above, mild erosion	Kaolinites
Landslips	Kaolinites, chlorites, montmorillonites, illites
Surface micro relief (gilgai)	Montmorillonites
Country rock type granitic	Kaolinites, micas
Country rock type basaltic, poorly drained topography	Montmorillonites
Country rock type basaltic, well drained topography	Kaolinites
Country rock type sandstones	Kaolinites
Country rock type mudstones and shales	Montmorillonites or illite, often soil salinity
Country rock type limestone	Alkaline montmorillonites and of very variable properties
Country rock type recent pyroclastics	Allophanes

Note: [(1)]The dominant component of the soil may be silt or sand sized, e.g. for granites, sandstones.

Table 7.5 Clay mineral identification from the soil profile (Ingles and Metcalf, 1972).

Inferences from the profile – Observation	*Dominant clay component*
Mottled clays, red-orange-white mottle	Kaolinites
Mottled clays, yellow-orange-gray mottle	Montmorillonites
Medium to dark gray and black clays	Montmorillonites
Brown and red-brown clays	Appreciable illite, some montmorillonite
White and light gray clays	Kaolinites and bauxites
Discrete micro-particles of high light reflectance (micas)	Micaceous soils
Discrete micro-crystals, easily crushed	Gypsum-rich soils, or (rarer) zeolites
Soft nodules, acid-soluble, disseminated	Carbonates
Hard nodules, red-brown	Ironstones, laterite
Extensive cracking, wide, deep and closely spaced at 5 to 6 cm or less	Calcium-rich illites and montmorillonites
Up to intervals of 30 cm and more	Illites
Open-textured friable loamy soils with appreciable clay content	Usually associated with carbonate, allophane or kaoline, but never montmorillonite and seldom illite
Open-textured friable loamy soils with appreciable clay content, black	Organic soils, peats
Open-textured friable loamy soils of low clay content	Carbonate, silts and sands
Wormy appearance on exposed pre-existing weathered profile	Montmorillonites, plus soil salinity
Relatively thin, strongly bleached horizon near the soil surface (up to 60 cm from the top)	Above the bleach, fine silt: below the bleach, dispersive clay. Probably a seasonal perched water table at the bleach level

7.7 USE OF DISPERSIVE SOILS IN EMBANKMENT DAMS

7.7.1 Problems with dispersive soils

Dispersive soils are a major contributing factor to internal erosion and piping failure of embankment dams, particularly for small homogeneous dams constructed without filters and often with poor construction supervision. This view is widely held, e.g. Sherard *et al.* (1976a, b, 1985a). However failures do occur in structures which are reasonably well engineered. The main contributory factors are:

- The presence of dispersive soils in the embankment or foundation.
- Poor compaction of the soil, i.e. to a low density ratio and/or dry of optimum water content.
- Poor compaction of soil around pipes or conduits which pass through the embankment.
- The presence of soil structure, e.g. root holes, fissures or cracks, in the soil in the dam foundation and with no adequate cutoff or erosion control measures.
- Erosion of dispersive or erodible embankment soils into open fractures in the rock in the sides of the cutoff trench.
- Poor cleanup of loose soil, grass etc from the cutoff foundation prior to placing earthfill.
- Cracking due to differential settlement or desiccation.
- Rapid filling of storages, giving insufficient time for cracks in the soil in the embankment to be sealed by the soil swelling or being wetted. The cracks may be due to desiccation during or after construction, differential settlement or hydraulic fracturing.

As indicated in ICOLD (1990) the majority of piping failures in dams constructed of dispersive soil occur on first filling, including cases when the reservoir has been raised after being at a given elevation for a period of time. The reasons for this include the presence of cracks and high permeability zones being present and "found out" on first filling and the cracks not having time to close as the soil around swells on saturation.

Wan and Fell (2002, 2004) have also shown that the rate of erosion of saturated clay soils is lower than partially saturated.

Piping failure can also be caused by introduction of water with low ionic concentration (i.e. 'fresh' water with a low salt content) into soils which were naturally saline, e.g. as shown in Figure 7.16. Ingles and Wood (1964) describe such a case. ICOLD (1990) indicates that virtually all failures occur in the embankment, not through soil in the foundation. However the authors have observed failures through the foundation in small farm dams.

Examples of piping failure of dams constructed using dispersive soils are given in Aitchison *et al.* (1963), Aitchison and Wood (1965), Cole and Lewis (1960), Sherard and Decker (1977), Cole (1977), Rallings (1960), Wagener *et al.* (1981), Bell and Maud (1994).

The authors' experience has been that piping failures usually occur in farm dams which have not been compacted other than by tracking of a bulldozer, but that even apparently well engineered structures have failed.

Examination of these cases shows failure to be related to the presence of dispersive soils (Emerson Class 1 or 2) compacted poorly (density ratio as low as 90% of standard compaction, as much as 2% to 4% below optimum water content, around outlet pipes through the embankment. In one case failure on first filling occurred despite the downstream slope of the dam being very flat (15H:1V). Sherard (1985a) describes a near piping incident of Wister Dam, where the hydraulic gradient was 1 in 50, with piping occurring over 200 metres from upstream to downstream.

A contributory factor also has been use of rough compacted surfaces on the concrete surrounding the outlet pipe, precluding proper compaction of the soil. In another case, cracking of soil by drying during construction around the trench into which the outlet pipe was placed and concreted was a contributory factor (see Figure 13.15).

It should however be emphasised that soils other than dispersive soils are subject to piping. Foster *et al.* (2000a) record that 9 out of 51 piping failures through embankment of large dams were recorded in dispersive soils, but note that the percentage may have been greater with the dispersive soils not necessarily having been recorded. Wan and Fell (2002, 2004) show dispersion class is only one of several factors which influence the erosion rate for a soil.

7.7.2 Construction with dispersive soils

As indicated in ICOLD (1990), safe dams can be built with dispersive clay provided certain precautions are taken. ICOLD (1990) indicates that the concern about the problems of dispersive soils and attention to precautions increase with dam size. While the authors understand the sentiment that failure of a large dam may be more important than a smaller one, they would argue that in reality it is easier to build in the necessary precautions to larger dams than smaller ones. In most cases, normal good practice for high consequence of failure dams will be all that is needed, e.g. well-designed filters, proper foundation preparation and good construction control. In small dams such measures may be regarded as uneconomic.

The following outlines the main precautions which should be adopted. These are based on ICOLD (1990) and the authors' own experience.

7.7.2.1 *Provide properly designed and constructed filters*

Sherard *et al.* (1984a, b) have proven conclusively that erosion of dispersive soils can be controlled by properly designed filters. If the guidelines outlined in Chapter 9 are followed, and filters are provided:

- within the embankment to control erosion of the Zone 1 earthfill;
- on the foundation, if it is dispersive or erodible soil, either as a horizontal drain or as a filter layer between the foundation and rockfill;
- around outlet pipes, as shown in Figure 13.16.

Then there should not be problems with piping failure.

As discussed in Chapter 9 if the soils are dispersive, more conservative criteria for selecting the grain size of the filter should be used, than if the soil is non erodible.

7.7.2.2 *Proper compaction of the soil*

Dispersive soils, particularly if being placed around outlet pipes, or at the contact between the earthfill and concrete structures, or if no filters are being provided (e.g. in a small low hazard dam), must be properly compacted, otherwise there is a high probability of piping failure.

ICOLD (1990) recommend compaction at a moisture content above optimum water content so as to avoid a flocculated soil structure and to avoid brittleness which will promote formation of cracks. They suggest that a permeability of lower than 10^{-7} m/sec is required.

The authors' opinion is that a water content between optimum and optimum plus 2% and a density ratio of greater than 98% (standard compaction) is desirable and that the water content should not be below optimum −1%. If the soil were at optimum +2% or even say optimum +3%, one might relax the density ratio requirement to 97%, the soil would probably be of low permeability and less likely to crack than if compacted at optimum −1%.

This will necessitate use of thin layers, particularly adjacent outlet pipes, where rollers may not be used in some cases. Supervision and testing need to be very thorough. Care must be taken to avoid drying of the surface of layers of earthfill, which could result in cracking, and a preferred path for piping failure to initiate.

Soils compacted to 98% density ratio dry o optimum minus 1% are in fact unlikely to suffer collapse settlement on saturation so are unlikely to result in a concentrated leak.

It should also be noted that good compaction alone does not guarantee that a dam will not experience cracking. This is discussed in some detail in Chapter 8.

7.7.2.3 *Careful detailing of pipes or conduits through the embankment*

Pipes through embankments should be avoided if possible since it is very difficult to ensure good compaction around the pipe, and differential settlement around the pipe can also lead to cracking of the soil. If pipes must be placed through an embankment then support the pipe on a concrete footing or better encase the pipe completely in concrete with sloping sides and use filters to surround the downstream end of the pipe as shown in Figures 13.18, 13.19 and 13.20.

It is considered that reliance only on good compaction with or without lime modification is a relatively high risk option and is not recommended.

7.7.2.4 *Lime or gypsum modification of the soil*

Most dispersive soils can be rendered non dispersive by addition of a small quantity of lime ($Ca(OH)_2$ or CaO) or gypsum ($CaSO_4$ or $CaSO_4(2H_2O)$).

This process is one of cation exchange, with the Ca^{++} ions exchangeable for Na^{++} ions. Laboratory tests can be carried out to determine the required amount of lime or gypsum, e.g. Sherard pinhole tests using soil with different amounts of lime or gypsum. Commonly one would require 2% to 3% lime, and would allow a margin of 1% or 2% above that indicated by the laboratory tests to allow for difficulty in mixing the lime with the soil. The lime should be mixed with the soil using a pulveriser. This breaks up the soil so that 80% to 90% of the particles are less than 25 mm diameter

and facilitates good mixing of the lime. The treatment of dispersive soils with lime is described in USBR (1998).

The authors note that, for some dams only, the downstream part of the core has been treated with lime or gypsum. This would leave the upstream dispersive soil with a low critical shear stress so erosion will initiate even in narrow cracks in the core. These particles will be transported through the lime treated core if the cracks persist as they would for cross-valley differential settlement for example. Given this it is preferred to treat the whole of the core.

7.7.2.5 *Sealing of cracks in the abutment and cutoff trench*

If the soil used for construction is dispersive, particular care must be taken to seal cracks in the cutoff foundation, and the sides of the cutoff trench so the soil will not erode into the cracks. Extensive use of slush concrete or shotcrete is likely. If the cutoff trench is in soil, it may be necessary to provide a filter on the downstream side as shown in Figure 10.23.

7.7.3 Turbidity of reservoir water

The presence of dispersive soil in the reservoir area, or in the catchment, can lead to turbidity of the water in the reservoirs. Grant *et al.* (1977) describe how turbidity in the water for Cardinia Creek Dam, Melbourne, was prevented by adding 6500 tonnes of gypsum to the water on first filling, at a concentration of 40 mg/litre of water. Again the process is one of Ca^{++} displacing the Na^{+} in the soil and makes it less dispersive. Gypsum was added to water as it entered the storage. Cardinia Creek Dam is an off river storage reservoir.

McDonald *et al.* (1981) describe how turbidity in Ben Boyd Dam, Eden NSW, was controlled by adding 1% by weight of gypsum of the soil depth in the reservoir area (27 tonnes per hectare). Gypsum was mixed in to 150 mm depth by disc harrow and compacted with a sheep's foot roller and gave initially clear water, but flood rains washed soil from the catchment and led to turbidity. The storage water was subsequently flocculated and clarified by adding 40 mg/l of alum ($Al_2(SO_4)_3$) and 20 mg/litre NaOH.

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Chapter 8

Internal erosion and piping of embankment dams and in dam foundations

8.1 THE IMPORTANCE OF INTERNAL EROSION AND PIPING TO DAM SAFETY

Internal erosion is an important potential failure mode for large and small dams, dykes and levees as shown by the statistics of historic of failures and incidents.

Based on the records of dam incidents and the dam register in ICOLD (1974, 1983a, 1995), Foster *et al.* (1998, 2000a) analysed the statistics of dam failures for large dams constructed between 1800 and 1986. Table 8.1 summarizes the results.

Foster *et al.* (1998, 2000a) also analysed information on accidents, and when these occurred. This is summarized in Tables 8.2 to 8.5. Table 8.6 summarizes the ICOLD (1995) data from 1970–1979 and 1980–1989.

It can be seen that internal erosion has been responsible for about half of embankment dam failures where the mode of failure is known. It is approximately equal in importance to failures by overtopping in floods due to inadequate spillway capacity and malfunction of gates and other outlets leading to overtopping.

By comparison embankment slides and failures due to earthquakes account for only 4% and 1.7% of embankment dam failures in operation respectively.

Overall about 1 in 180 embankment dams in the database have failed by internal erosion and 1 in 100 embankment dams have experienced an internal erosion accident.

The largest number of failures occurs in the embankment. Nearly half of these are associated with conduits that penetrate through the embankment, or walls which support the embankment.

There are many more accidents than failures. As defined by ICOLD, these are incidents that have been prevented from becoming failures by immediate remedial measures, including possibly drawing down the water in the reservoir. There have been a similar number of accidents relating to internal erosion in the foundation as in the embankment.

For all internal erosion modes about two thirds of all failures and about half of accidents occur on first filling or in the first 5 years of operation. This means however that around half of all incidents have occurred in dams after 5 years of operation.

Foster *et al.* (1998, 2000a) found that nearly all internal erosion failures in the embankment occurred when the reservoir level was at the highest level ever, or within 1 metre of that level. For internal erosion in the foundation the reservoir level was not as important.

Table 8.1 Overall statistics of embankment dam failures, large embankment dams, up to 1986 and excluding dams constructed in Japan pre 1930 and in China (adapted from Foster *et al.*, 1998, 2000a).

	Number of cases		*% failures (where known)*		*Average probability of failure (10^{-3})*	
Mode of failure	*All failures*	*Failures in operation*	*All failures*	*Failures in operation*	*All failures*	*Failures in operation*
Inadequate spillway capacity	46	40	36	34	4	3.5
Malfunction of gate	16	15	12	13	1.5	1.5
Subtotal overtopping & appurtenant failures	62	55	48	47	5.5	5
Internal erosion through embankment	39	38	30	33	3.5	3.5
Internal erosion through foundation	19	18	15	15	1.5	1.5
Internal erosion from embankment into foundation	2	2	1.5	1.5	0.2	0.2
Subtotal internal erosion[1]	59	57	46.5	48.5	5	5
Downstream slides	6	4	5	3.5	0.5	0.4
Upstream slides	1	1	1	1	0.1	0.1
Subtotal slides	7	5	6	4.5	0.6	0.5
Earthquake/liquefaction	2	2	1.5	1.5	0.2	0.2
Unknown mode	8	7				
Total no. of failures[1]	136	124			12	11
Total no. of failures (where mode of failure known)	128	117				
No. of embankment dams	11192	11192				

Notes: [1]Subtotals and totals do not necessarily sum to 100% as some failures were classified as multiple modes of failure.

Table 8.2 Historical frequencies of failures and accidents in embankments of large dams constructed from 1800 to 1986, excluding dams constructed in Japan pre 1930 and in China (adapted from Foster *et al.*, 1998, 2000a).

Case	*Total*	*In embankment*	*Around conduits and adjacent to walls*
Internal erosion failures	36	19	17
Internal erosion accidents	75	52	23
Seepage accidents with no detected erosion	36	30	6
Total number of failures and accidents	146	101	46
Population of dams	11192	11192	5,596
Historical frequency for failures and accidents	0.013	0.009	0.0082
Proportion of failures and accidents on first fill	36%		
Proportion of failures and accidents after first fill	64%		
Historical frequency for first fill		0.0032	0.0030
Historical frequency after first fill		0.0058	0.0052
Historical annual frequency after first fill		2.2×10^{-4}	2.0×10^{-4}

Table 8.3 Time of incident after construction – internal erosion through the embankment incidents (adapted from Foster *et al.*, 1998, 2000a).

	Number of cases		*% of cases (where known)*	
Time of incident	*Failures*	*Accidents*	*Failures*	*Accidents*
During construction	1	0	2	0
During first filling	24	26	48	26
After first filling and during first five years	7	13	14	13
After first five years	18	60	36	61
Unknown	1	3	–	–
Total number of incidents	51	102	100	100

Table 8.4 Time of incident after construction – internal erosion through the foundation incidents (adapted from Foster *et al.*, 1998, 2000a).

	Number of cases		*% of cases (where known)*	
Time of incident	*Failures*	*Accidents*	*Failures*	*Accidents*
During construction	1	0	5	0
During first filling	4	23	20	30
After first filling and during first five years	10	19	50	24
After first five years	5	36	25	46
Unknown	1	7	–	–
Total number of incidents	21	85	100	100

Table 8.5 Time of incident after construction – internal erosion from the embankment into the foundation incidents (adapted from Foster *et al.*, 1998, 2000a).

	Number of cases		*% of cases (where known)*	
Time of incident	*Failures*	*Accidents*	*Failures*	*Accidents*
During construction	0	0	0	0
During first filling	2	6	50	20
After first filling and during first five years	2	8	50	27
After first five years	0	16	0	53
Unknown	0	1	–	–
Total number of incidents	4	31	100	100

Table 8.6 Failure statistics for embankment dams excluding China from 1970 to 1989 (adapted from Foster *et al.*, 1998, 2000a).

Modes of failure	*Ratio of failures to the number of dams 1970–1979*	*Ratio of failures to the number on dams 1980–1989*
Internal erosion	0.0020	0.0016
Overtopping and appurtent structure failure resulting in overtopping	0.0026	0.0019
Sliding	0.0004	0.0001

There is evidence from the ICOLD data in Table 8.6 that the later statistics are better reflecting improved design and construction methods. However the problems are ongoing.

There has not been a systematic study of international failure and accident statistics since the ICOLD (1995) study. However there have been studies for individual countries.

Brown and Gosden (2004, 2008) estimated that in the United Kingdom's population of about 2,500 dams registered under the Reservoirs Act, there are about 1,600 incidents for every recorded failure. About 60% of the incidents appear to be related to internal erosion, with typically two serious erosion incidents a year. It should be noted that the definition of incident used in this study is not the same as in ICOLD (see Glossary). It includes incidents which are unlikely to have resulted in failure. The "severe incidents" however would classify as ICOLD accidents.

Engemoen and Redlinger (2009) and Engemoen (2011) carried out a detailed analysis of internal erosion incidents for the Bureau of Reclamation 220 embankment dams. They found that internal erosion can occur at any time in a dam's life; that successful performance of a dam does not preclude it from having an internal erosion incident; that 99 dams had experienced incidents, of which only one was a failure; 53 involved definite particle transport and the remainder were excessive seepage or sand boils. Of the 99 incidents 9 were in the embankment, 70 in the foundation, 6 were from embankment into a conduit, 5 were into or along a conduit and 11 were into toe or under drains. They developed annualized statistics of failure and incident from these data.

8.2 DESCRIPTION OF THE INTERNAL EROSION AND PIPING PROCESS

8.2.1 The overall process leading to failure of a dam

Failures and incidents by internal erosion of embankment dams and their foundations are categorized into three general failure modes, which are:

- *Internal erosion through the embankment*, which includes internal erosion associated with through-penetrating structures, such as conduits associated with outlet works, spillway walls or other adjoining a concrete gravity structure supporting the embankment.
- *Internal erosion through the foundation* and
- *Internal erosion of the embankment into or at the foundation*. Including (a) seepage through the embankment eroding material into the foundation, or (b) seepage in the foundation at the embankment contact eroding the embankment material.

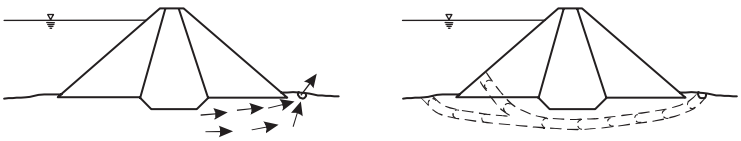
The process of internal erosion and piping may be broadly broken into four phases:

- initiation of erosion
- continuation of erosion
- progression to form a pipe
- Initiation of a breach.



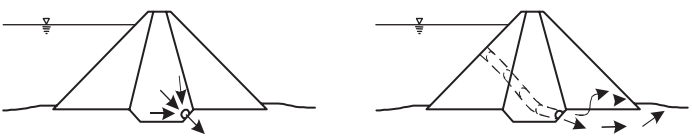
INITIATION	→ CONTINUATION	→ PROGRESSION	→ BREACH
Concentrated leak forms, erosion initiates along walls of crack	Continuation of erosion	Enlargement of concentrated leak	Breach mechanism forms

(a) Internal erosion in the embankment initiated by erosion in a concentrated leak



INITIATION	→ CONTINUATION	→ PROGRESSION	→ BREACH
Leakage exits from the foundation and backward erosion initiations	Continuation of erosion	Backward erosion in progresses to form a pipe	Breach mechanism forms

(b) Internal erosion in the foundation initiated by backward erosion piping



INITIATION	→ CONTINUATION	→ PROGRESSION	→ BREACH
Leakage exits the core into the foundation and backward erosion initiates as core erodes into the foundation	Continuation of erosion	Backward erosion progresses to form a pipe. Eroded soil is transported in the foundation	Breach mechanism forms

(c) Internal erosion from embankment to foundation initiated by backward erosion piping

Figure 8.1 Models for the development of failure by internal erosion (Foster and Fell, 1999).

This is shown in Figure 8.1(a) for internal erosion through the embankment initiated by a concentrated leak. Similar processes apply for piping through the foundation, and from the embankment to the foundation and are shown in Figure 8.1(b) and (c).

For practical purposes it is better to refine these processes to the following generic sequence of events (Fell *et al.*, 2008):

↳ Reservoir Rises
↳ Initiation – Flaw exists[1][2]
↳ Initiation – Erosion starts
↳ Continuation – Unfiltered or inadequately filtered exit exists (consider: no erosion/some erosion/excessive erosion/continuing erosion)
↳ Progression – Roof forms to support a pipe
↳ Progression – Upstream zone fails to fill crack
↳ Progression – Upstream zone fails to limit flows
↳ Intervention fails
↳ Dam breaches (consider all likely breach mechanisms)
↳ Consequences occur

As discussed in Section 8.12.6, this can be developed into an event tree structure for quantitative risk analyses.

8.2.2 Initiation of internal erosion

Initiation and progression of internal erosion may occur by:

- Concentrated leak.
- Backward erosion.
- Suffusion.
- Contact erosion.

The nature of the soil in the dam or foundation determines its mechanism of internal erosion:

- Non plastic soils such as silts, sands, silty sands, and silt, sand, gravel mixtures are subject to backward erosion, contact erosion or suffusion depending on their particle size distribution.
- Plastic soils, such as clays, clayey sands, and clayey sandy gravels are subject to concentrated leak erosion and contact erosion. Backward erosion and suffusion cannot occur in these soils under the gradients normally experienced in dams and their foundations but may occur if local gradients are very high.
- Dispersive plastic soils are soils in which, because of their clay mineralogy and the water chemistry, erosion will initiate in cracks or concentrated leaks under very low hydraulic stresses and gradients.

These are discussed further in Sections 8.3 to 8.6.

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[1] A "flaw" is a continuous crack, high permeability or poorly compacted zone in which a concentrated leak may form.
[2] For Backward Erosion Piping (BEP) no flaw is required but a continuous zone of cohesionless soil in the embankment or foundation is required.

8.2.3 Continuation of erosion

Continuation of erosion relates to whether filters or transition zones within the dam will arrest the erosion process.

Depending on the gradation of the soil and the filter, the erosion will either:

- not continue (no-erosion), or
- stop after only minor erosion (some erosion), or
- stop only after a significant amount of erosion (excessive erosion), or
- will continue (continuing erosion).

This is discussed in Chapter 9.

8.2.4 Progression of erosion

Progression is the phase of internal erosion where:

a. For *concentrated leak erosion*, the erosion in the crack or concentrated leak leads to the development of a pipe.
b. For *backward erosion* the erosion process extends upstream from the point of initiation, and a network of small erosion channels forms beneath the soil or embankment providing the roof to the erosion pipes. If these small erosion channels reach the reservoir or river, then a pipe forms.
c. For *suffusio*n, some of the finer fraction is eroded leaving the coarse matrix of the soil. No pipe is formed but the permeability of the soil may be increased significantly.
d. For *contact erosion* the erosion of the finer soil into the coarser soil or into open defects in a rock mass continues. This may in particular cases lead to development of a pipe in the finer soil.

The development of progression depends on whether the soil within which the pipe is forming, or overlying a concentrated leak or contact erosion will support the roof of the pipe; and whether the upstream zone may limit flows to reach an equilibrium condition, or soil from the upstream zone washes into the eroding soil and stops the process.

This is discussed further in Section 8.8.

8.2.5 Detection and intervention

The likelihood that a particular failure path can be detected and, if so, whether it is possible to intervene (e.g. by lowering the reservoir level) or carry out repairs to prevent the dam breaching is usually best considered as two questions:

1. Will this failure path be detected?
2. Will intervention and repair be possible in the time available?

This is discussed further in Section 8.9.

8.2.6 Breach

Potential breach mechanisms are:

– Gross enlargement of the pipe
– Unravelling of the toe
– Crest settlement, or sinkhole on the crest leading to overtopping
– Instability of the downstream slope.

8.3 CONCENTRATED LEAK EROSION

8.3.1 The overall process

Whether erosion will initiate in concentrated leaks in a dam or its foundation depends on whether the stresses in the dam will result in a crack or low stress zone subject to hydraulic fracture and geometrical and hydraulic criteria within the crack:

- Whether there is a crack (or flaw) below the reservoir level formed by differential settlement during or after construction, hydraulic fracture, desiccation, or collapse of a poorly compacted layer of soil in the embankment or around a conduit sited to pass through the embankment.
- Given there is a crack, whether the forces imposed on the sides of the crack by the water flowing through the crack are sufficient to initiate erosion.
- Whether the flow velocity is sufficient to transport the detached particle in the crack (Figure 8.2).

Whether in the absence of filters or where filters are too coarse, the erosion will progress to form a pipe depends on:

- Whether the water flowing through the crack will cause the soil on the sides of the crack to swell, closing the crack, or reducing the width so the forces imposed on the sides of the crack by the water flowing through the crack are insufficient for erosion to progress.

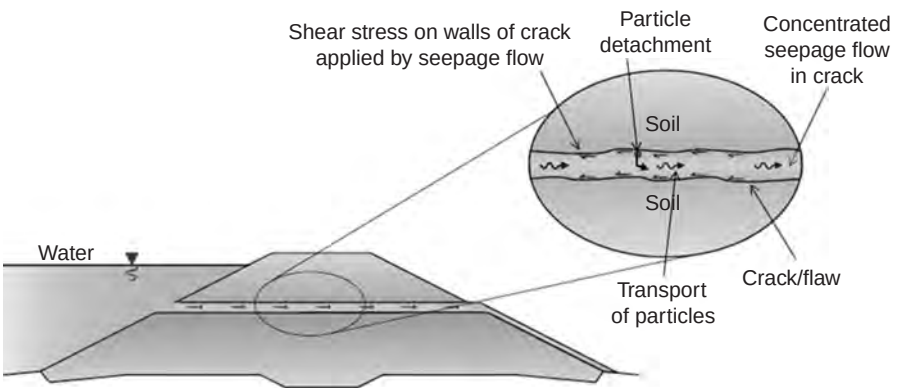


Figure 8.2 Detachment and transport of particles in concentrated leak erosion.

- Whether the soil will hold open the pipe ("will it hold a roof").
- Whether upstream zones will limit the erosion process or crack stopping occurs.

Also important is the rate of erosion and enlargement of the pipe because this influences whether the leak can be detected and intervention taken to lower the reservoir or stop the erosion process before breach occurs.

Fell *et al.* (2008), Fell and Fry (2013) and Bonelli *et al.* (2013) provide additional information.

Hydraulic fracture can occur in the embankment or foundation when the water pressure ($\gamma_w \cdot h$) at a particular location exceeds the sum of the minor principal total stress (σ_3) and the tensile strength (σ_t)

$$\gamma_w \cdot h \geq \sigma_3 + \sigma_t$$

Figure 8.3 shows an example. An initial opening forms on the upstream side of the core due to a defect or crack. Water pressure in the initial water filled crack increases, for example due to increasing reservoir level. If the water pressure in the crack exceeds the sum of the minor principal stress and tensile capacity of the soil at the crack tip, then the crack is jacked open and it extends further into the core. In essence, the process progresses from upstream to downstream.

Hydraulic fracture can also occur due to use of high pressures when drilling with water or air, or by pressure grouting.

8.3.2 Situations where cracking and low stress zones may be present in an embankment or the foundation

8.3.2.1 *Cracking and hydraulic fracture due to cross valley differential settlement of the core*

As an embankment dam is constructed the partially saturated compacted soil in the embankment consolidates and settlement occurs. Where the valley sides are steep and/or have steps in the profile, such as shown in Figure 8.4, differential settlements

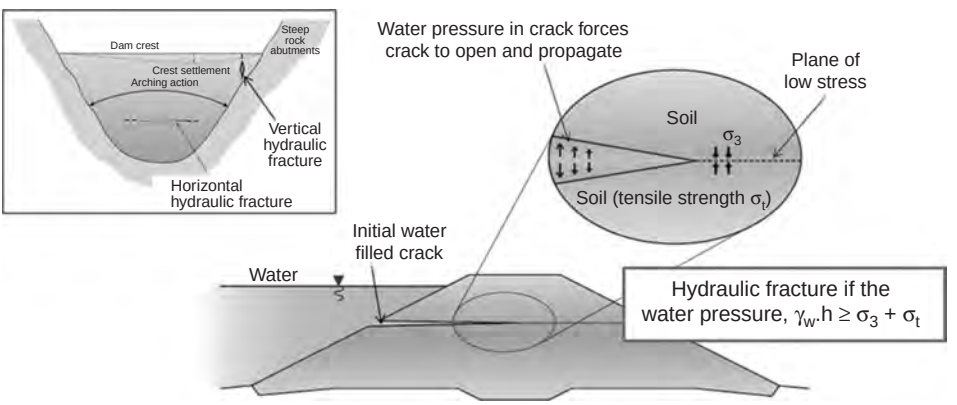


Figure 8.3 Mechanism of formation of hydraulic fractures in embankment dams.

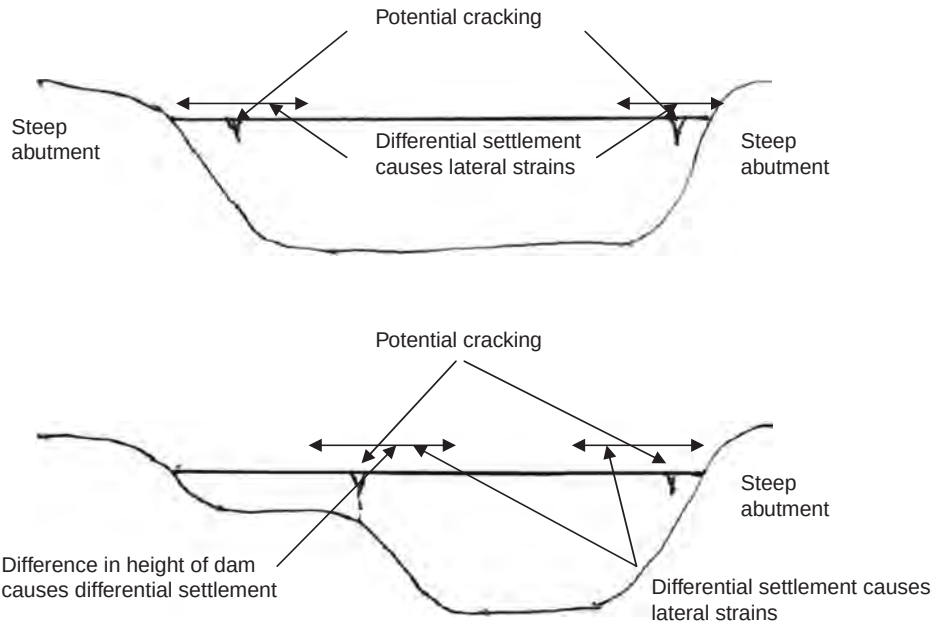


Figure 8.4 Situations in which cross-valley differential settlement may lead to cracking or lateral strain and low stress zones subject to hydraulic fracture.

occur due to the variations in the height of the embankment, and these can lead to tensile or low stress zones in which cracks may form. This has been recognised for some time by Sherard (1973, 1985a, 1986), Høeg *et al.* (1998), Kjaernsli *et al.* (1992) and others. Hydraulic fracture through these low stressed zones as the dam is filled or under flood conditions is an associated phenomenon as discussed by Sherard *et al.* (1972) and Sherard (1985a, 1986). For most dams, about 80% to 90% of the total settlement occurs during construction (Hunter, 2003; Hunter and Fell, 2003d), so the stresses set up in the construction phase largely control the likelihood of low stress zones and cracking.

It should be recognized that these stresses are inevitable even in well-designed and constructed dams and are not caused by poor design or construction practice.

Bui *et al.* (2004, 2005) carried out extensive numerical modelling to determine the conditions which are conducive to cracking and low stress zones due to cross valley differential settlements. The modelling was validated by carrying out numerical modelling of cases where cracking had been observed or internal erosion and piping had occurred.

Figures 8.5 and 8.6 show examples of the stresses calculated for valley sections with and without a step in the abutment profile. Also shown are the lateral displacements (as a ratio of embankment height) which cause the low stress zones. It should be noted that the magnitudes of the tensile stresses are not accurately modelled, but the extent of low stresses is a reasonable representation.

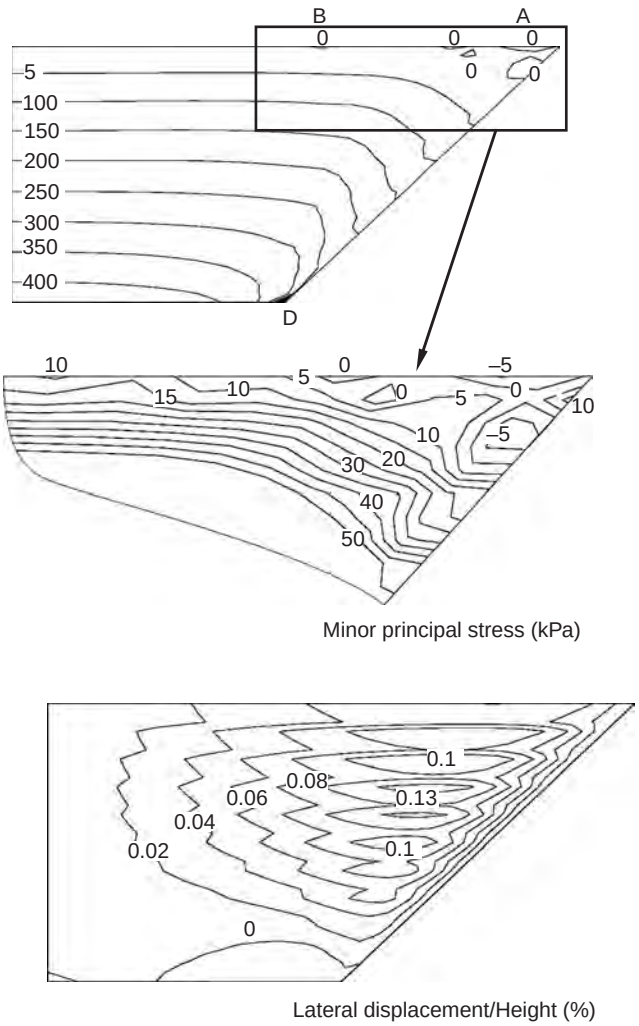


Figure 8.5 Results of Mohr Coulomb cross-valley model without a step in the foundation profile (Bui *et al.*, 2004).

Figure 8.7 shows the stresses during the stages of construction of an embankment. It can be seen that low stress zones form over the step but these go back into compression as the embankment is raised. The modelling shows cumulative net extension strains in such areas even though they go back into compression.

Based on cracking observed at Buffalo dam (Newman and Foster, 2006) and Eppalock dam (Davidson *et al.*, 2001) it is considered wise to assume cracking or low stress zones conducive to hydraulic fracture may persist down to the step in the foundation profile.

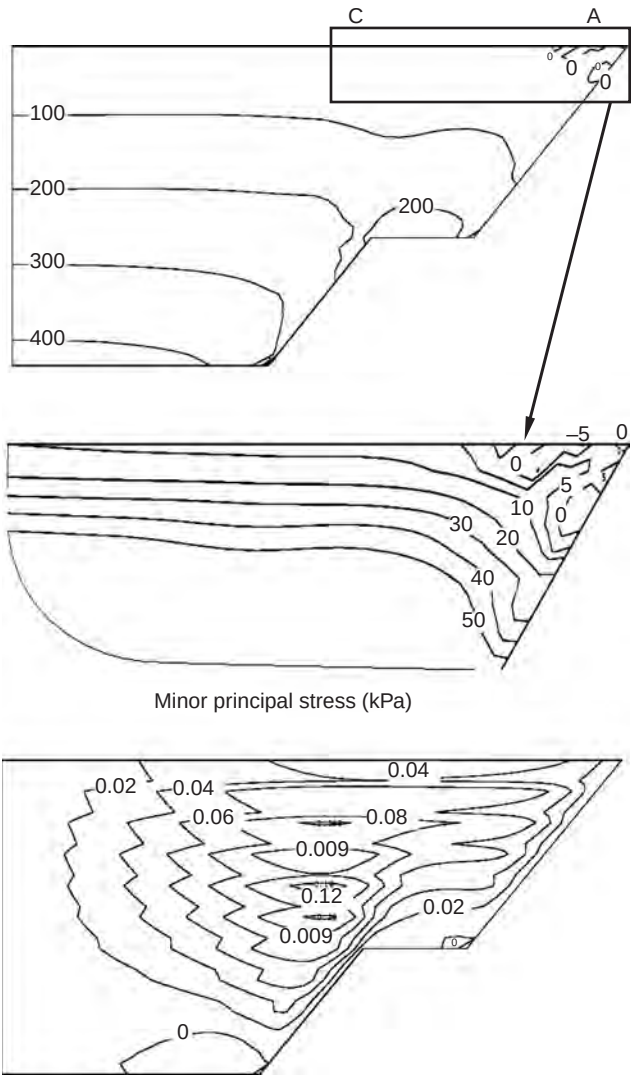


Figure 8.6 Results of Mohr Coulomb cross-valley model with a step in the foundation profile (Bui *et al.*, 2004).

Bui *et al.* (2004, 2005) have shown that the shapes of steps in the abutment profile are a secondary factor to the basic geometry.

As shown by the performance of Tullaroop dam (Hunter *et al.*, 2007) cracking may occur many years after construction of the dam as the built in stresses are relieved; in that case by record low reservoir levels and prolonged drought.

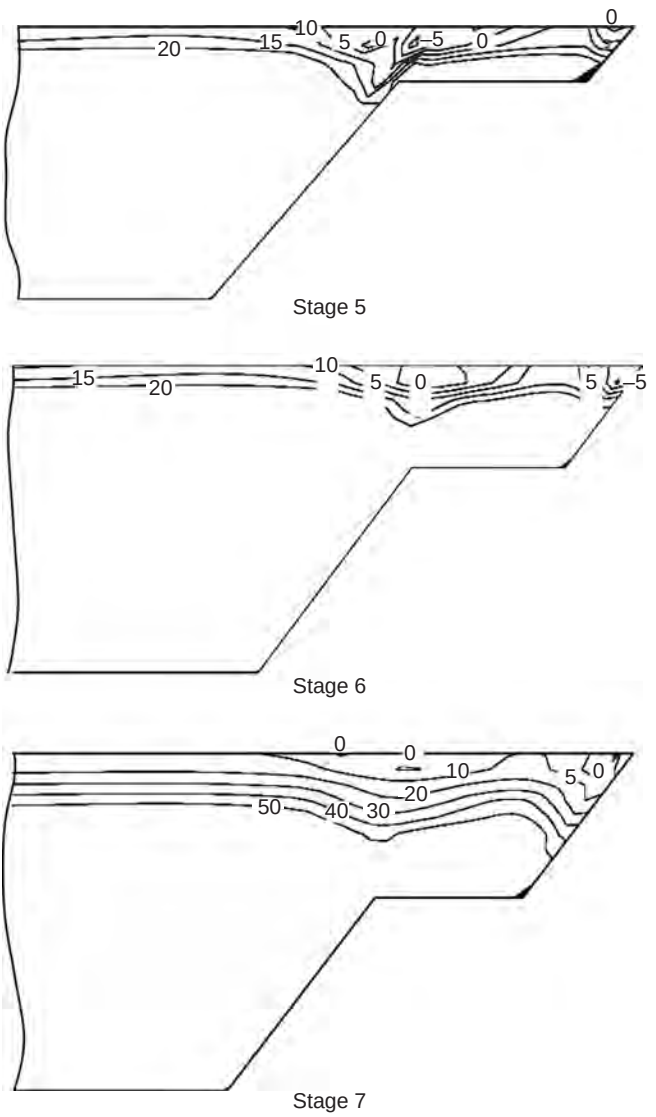
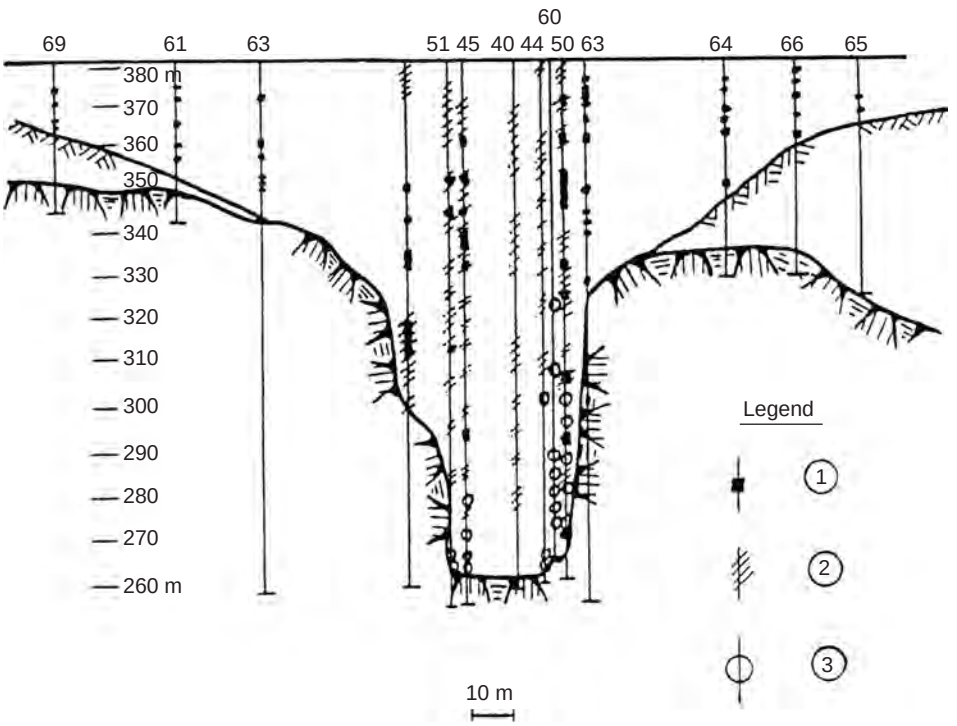


Figure 8.7 Low stress zones formed in an embankment dam during construction (Bui *et al.*, 2004).

8.3.2.2 *Cracking and hydraulic fracture due to cross valley arching*

If the valley in which the dam is constructed is narrow and steep, cross-valley arching can occur and the vertical stresses are shed onto the sides of the valley. This can lead to a situation where hydraulic fracture can occur. Fell *et al.* (2008) suggest that cross-valley arching is most likely to occur if the width of the valley base is less than a quarter



(1) Loose or soft zones as determined by ease of drilling, sampling, or casing advance without driving during drilling.
(2) Zones of relatively low density as indicated by geophysical down-hole logging.
(3) Water-producing or water-loss zones (also mud-loss zones in RD-60).

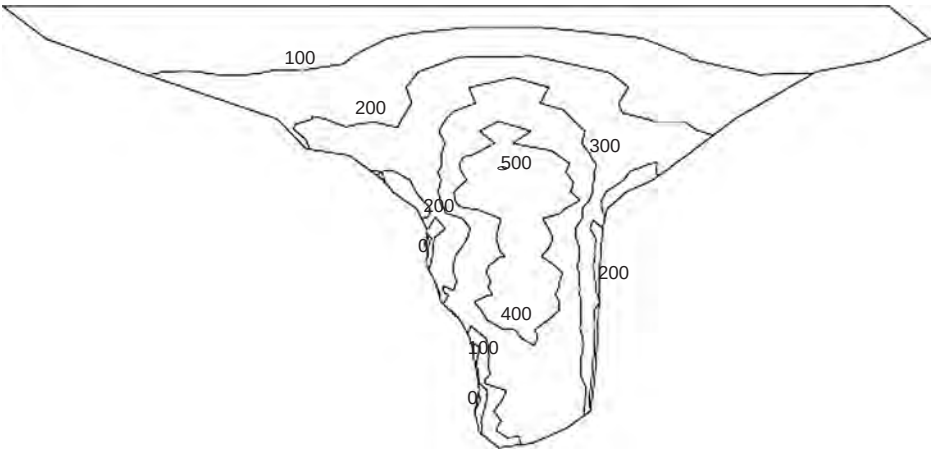
Figure 8.8a Mud Mountain dam valley section showing loose, soft and low density zones (Graybeal, 1988).

of the dam height and the valley sides are steeper than about 60 degrees. It is unlikely to be an issue if the width of the valley base is greater than three quarters the dam height and the valley sides are flatter than 45 degrees.

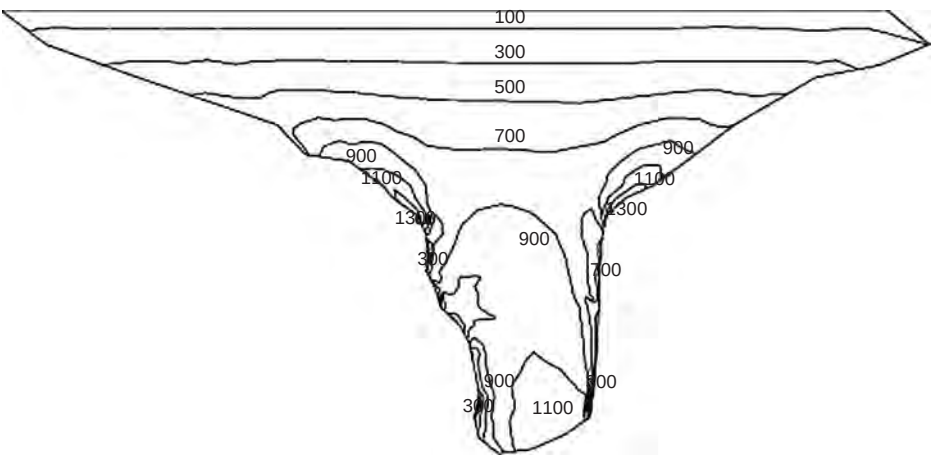
Figure 8.8b shows the results of cross valley numerical modelling by Bui *et al.* (2005) of the 120 m high Mud Mountain dam which experienced internal erosion due to hydraulic fracture resulting in loose and soft zones as shown in Figure 8.8a.

The modelling estimates minor principal stresses and vertical stresses much lower than the stresses calculated from the depth of fill, and lower than the pore pressures which develop as the reservoir fills, so hydraulic fracture is predicted which is consistent with the field behaviour.

Eckerlin (1992, 1993), Graybeal (1988) and Graybeal and Levallois (1991) give details of the internal erosion and remedial measures.



(a) Minor principal sress (kPa)



(b) Vertical stress (kPa)

Figure 8.8b Mud Mountain dam – minor principal stress and vertical stress distributions (kPa) (Bui *et al.*, 2005).

8.3.2.3 *Cracking and hydraulic fracture due to differential settlement in the foundation under the core*

Figure 8.9 which is adapted from Sherard *et al.* (1963) shows foundation conditions which are likely to lead to differential settlement and cracking or low stress zones conducive to hydraulic fracture. Zones which may lead to differential settlement greater than 0.5% of the dam height, with steep changes in the foundation profile ($\alpha > 45$ degrees) are most likely to suffer cracking and hydraulic fracture. Differential settlements of less than 0.2% of the dam height spread over some distance ($\alpha < 30$ degrees) are unlikely to lead to cracking and low stress zones.

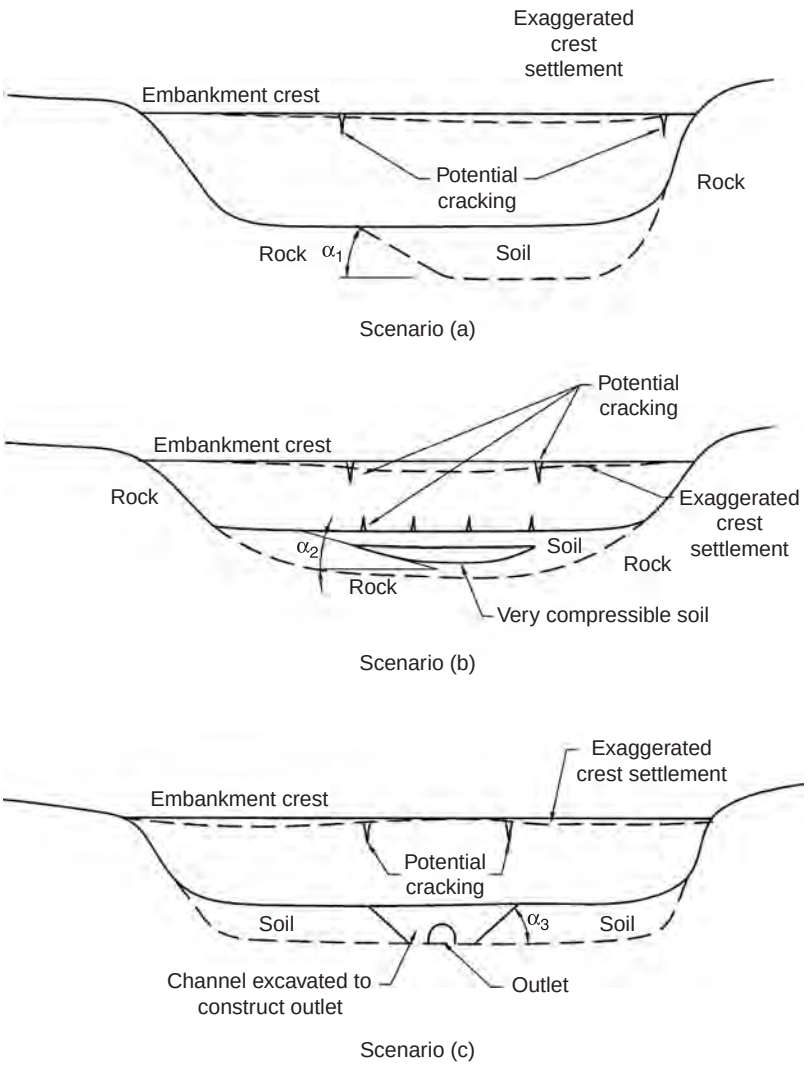


Figure 8.9 Situations which may cause differential settlement in the foundation of dams leading to cracking, lateral strains and low stress zones subject to hydraulic fracture (Fell *et al.*, 2008, based on Sherard *et al.*, 1963).

8.3.2.4 *Cracking and hydraulic fracture due to small scale irregularities in the foundation profile under the core*

Small scale irregularities in the foundation of the core can lead to cracking or low stresses conducive to hydraulic fracture, e.g. NGI (1984). For cracking or low stresses to occur, the small scale irregularities need to be persistent over all or most of the distance across the core and have steps greater than about 3% to 5% of the embankment height.

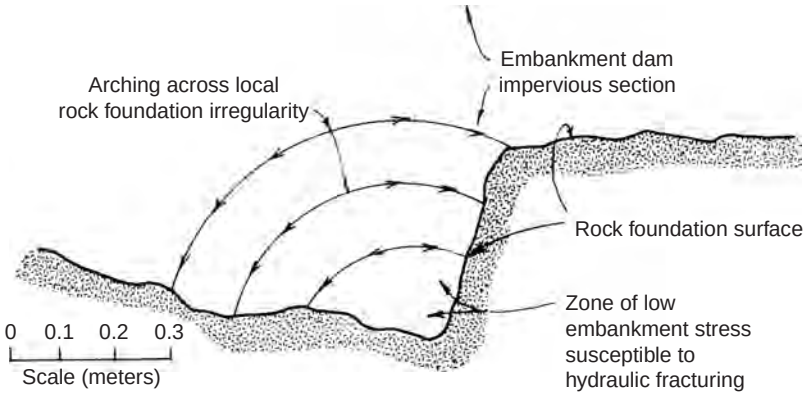


Figure 8.10 Zones of low embankment stress caused by localised arching adjacent to small scale irregularities in the foundation rock allowing hydraulic fracture (Sherard, 1985).

Figure 8.10 shows the mechanics of the process.

Figure 8.11 shows the results of 3D numerical analyses of deformations and stresses for Buffalo Dam. (Newman and Foster, 2006), this set of diagrams shows the low minor principal stresses over the bench formed by the haul road which is shown in Figure 8.13, and the vertical and lateral deformations. The crack formed at the edge of the bench was up to and extended over the full width of the core. The crack was open at the top of the core (about 40 mm wide) and then transitioned with depth into a softened zone containing cracks. Probing indicated this crack probably extended several metres to below reservoir full supply level. Figure 8.12 shows the cracking schematically.

Figure 8.14 shows the development of cracking and softened zones over steps in slope correction concrete at Matahina Dam (Gillon, 2007). These resulted in erosion of the core and sinkhole development. A similar detail on the other abutment also resulted in internal erosion and sinkhole development.

8.3.2.5 *Cracking due to lack of support for the core by the shoulders of the embankment*

Figure 8.12 shows schematically longitudinal cracking which occurred in Buffalo dam due to lack of support for the core by the loose dumped rock fill (Newman and Foster, 2006). These cracks which were up to 40 mm open at the top of the core can join to form a concentrated leak path through the core. Similar cracking has been observed in a number of central core earth and rock fill dams.

8.3.2.6 *Cracking and hydraulic fracture due to arching of the core onto the shoulders of the embankment*

As detailed in Bui *et al.* (2004, 2005) and Fell *et al.* (2008) this is most likely to be a problem for cores which are very narrow-core width less than 0.25 the embankment height and for soils subject to collapse compression on saturation (poorly compacted

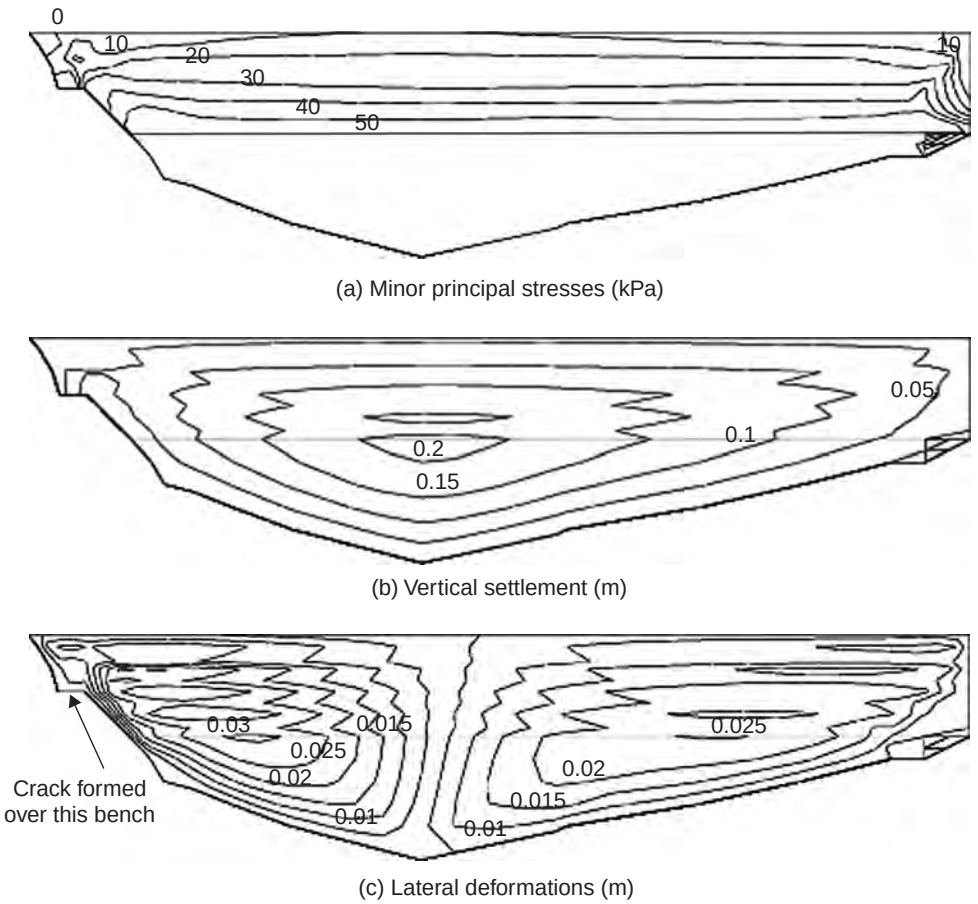


Figure 8.11 Results of deformation analysis for Buffalo Dam (Bui *et al.*, 2005).

soil placed dry of optimum moisture content). It is unlikely to be a problem for cores which are wider than 0.5 to 1.0 the embankment height and the core is well compacted at around optimum moisture content. It is not sufficient to cause hydraulic fracture to just have stiff shoulders and a less stiff core. There needs to be differential movement of the core after construction.

8.3.2.7 *Crack or gap adjacent to a spillway or abutment walls and where concrete dams abut embankment dams*

Cracking or a gap may form adjacent to walls due to the earthfill settling away from the wall during and after construction. Figure 8.15 shows some situations where this is likely to occur. The designer should also be mindful of the chances of low stress zones developing just a short distance in from a sloping wall surface in much the same way as is the case at steep abutments.

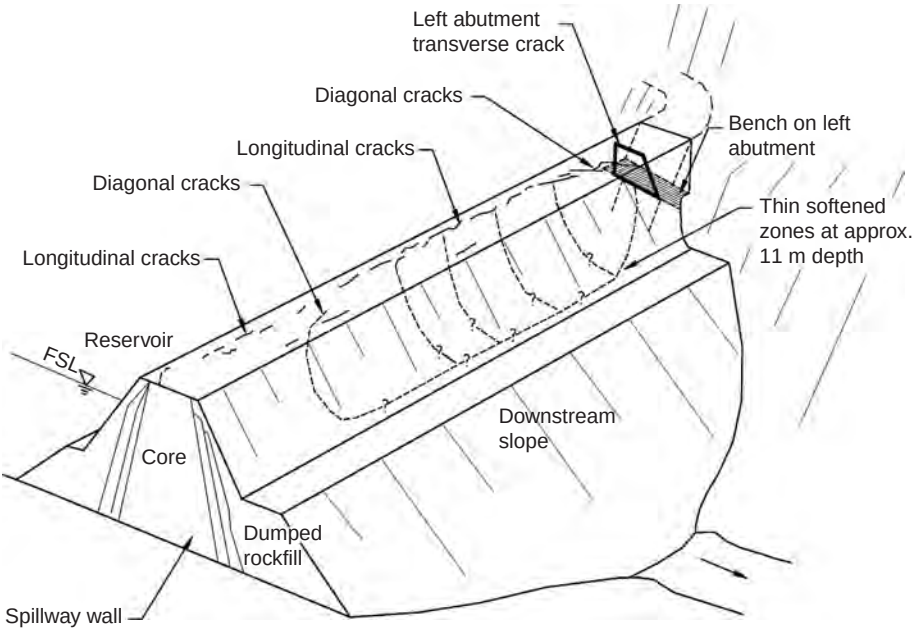
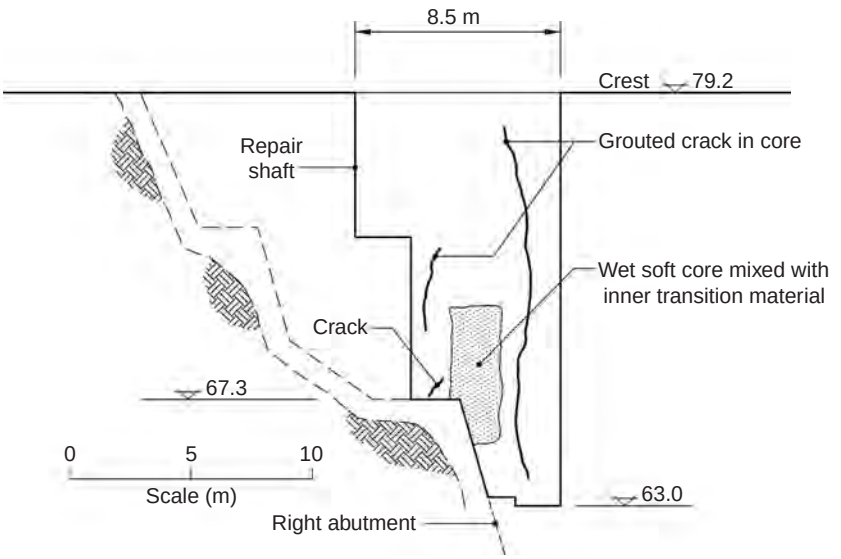


Figure 8.12 Sketch showing transverse and longitudinal cracking in Buffalo Dam (Newman and Foster, 2006).



(a) (b)

Figure 8.13 Buffalo Dam. (a) Bench in the abutment over which transverse cracking occurred and (b) Transverse crack (Courtesy of Goulburn-Murray Water and URS Australia).



(a) Section of core over slope correction concrete showing cracking and softened zones



(b) Photograph of abutment showing slope correction concrete

Figure 8.14 Cracking of the core on Matahina Dam (Gillon, 2007).

Well designed walls with uniform contact slopes flatter than about 0.25H:1V are unlikely to have gaps form. Vertical or over-hanging walls are likely to have gaps.

Cracking and gaps may form due to deformations of flexible and/or under-designed retaining walls (e.g. designed for active rather than at-rest earth pressures).

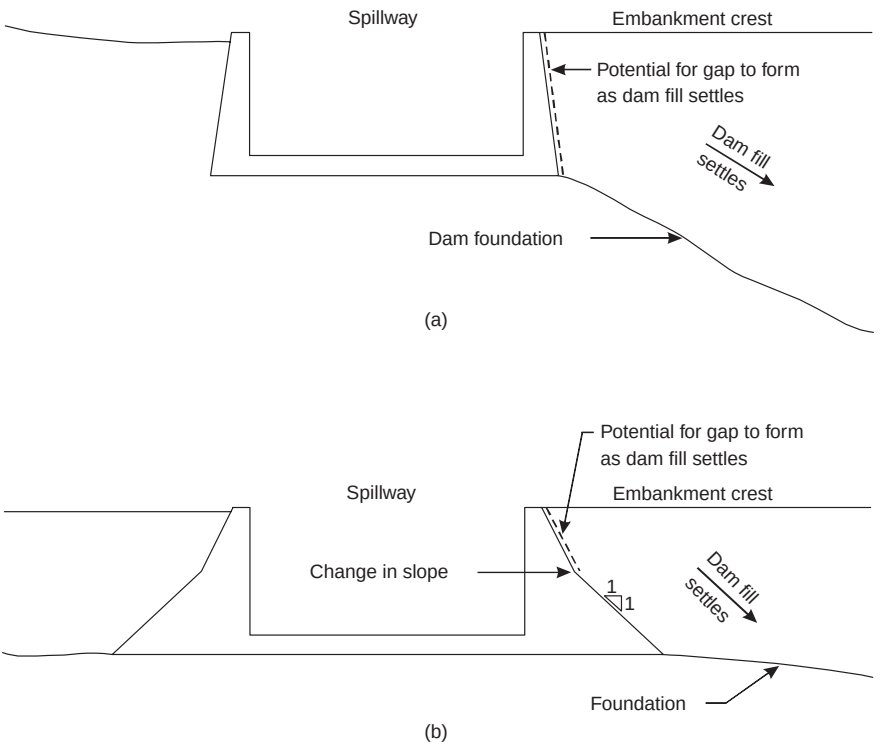


Figure 8.15 Situations where a gap may form between the dam fill and spillway wall (a) Steep foundation adjacent spillway wall; (b) Change in slope of the retaining wall (Fell *et al.*, 2005).

Where there is a wrap-around junction between a concrete gravity dam and an embankment dam as shown in Figure 8.16 differential settlements similar to those described above may occur and may result in cracks between the core and the concrete.

8.3.2.8 *Crack or hydraulic fracture in poorly compacted layers in the embankment*

It is well documented (e.g. Sherard 1973, Foster *et al.* (2000a) that internal erosion and piping occurs in poorly compacted cohesive soils. This is particularly so for dispersive soils. The mechanism is potentially of two types as shown in Figure 8.17.

- The soil behaves as a series of clods with openings between the clods in which water passes.
- The soil collapses on saturation forming a flaw (open pathway) in which the water can flow with relative ease.

This is most likely where there is poorly compacted soil against a conduit but is possible within layers of soil. Sherard (1973) gives examples of this.

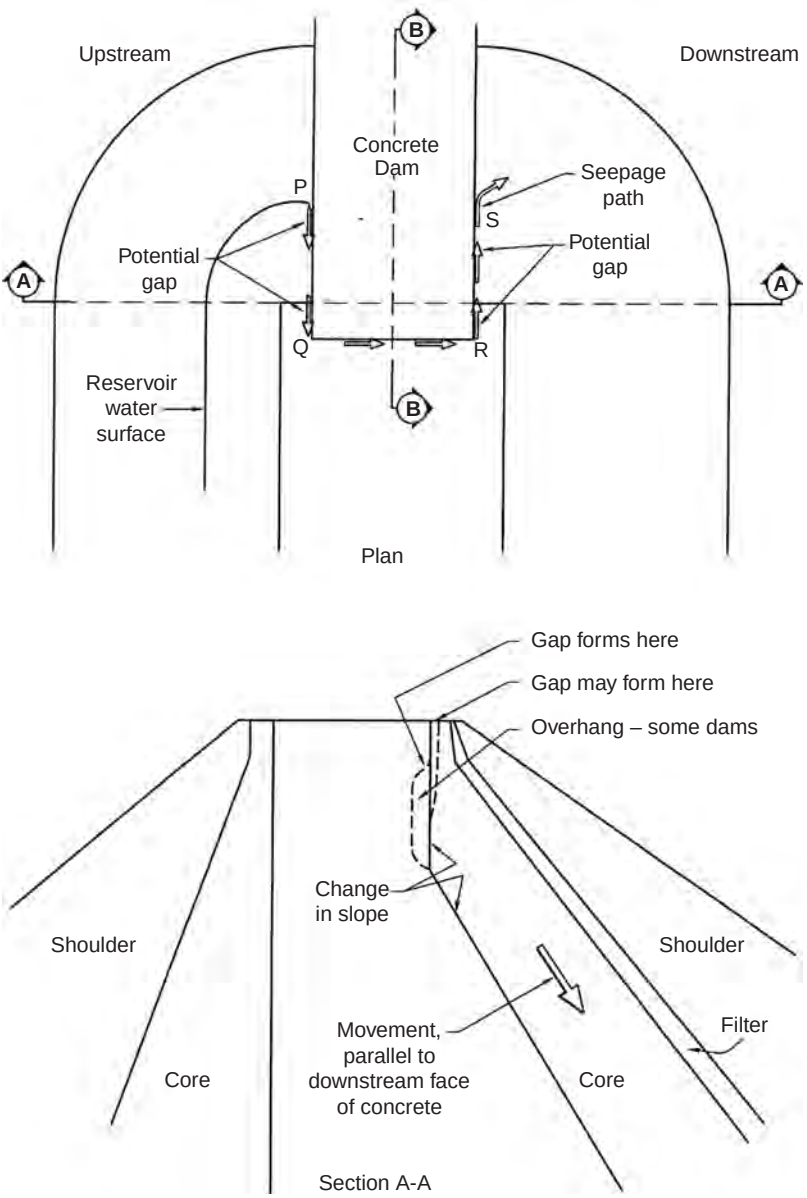


Figure 8.16 Wrap around details for connection of embankment dam to concrete gravity dam.

The opening of the crack resulting from collapse settlement can be estimated by estimating the thickness of the poorly compacted layer and using Table 8.8 to estimate the amount of collapse settlement depending on the degree or type of compaction.

The authors are aware of cases where a gap has been seen in test pits and/or zero Cone Penetration Test cone resistance value occurs just below the phreatic surface.

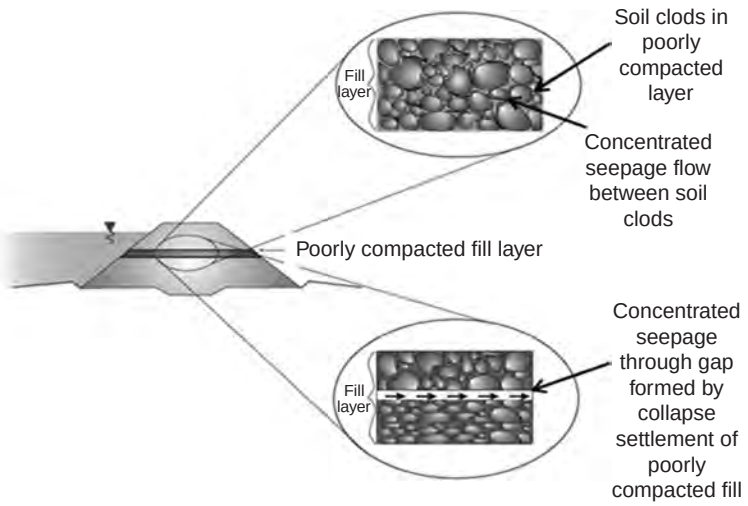

Soil clods in poorly compacted layer
Fill layer
Concentrated seepage flow between soil clods
Poorly compacted fill layer
Concentrated seepage through gap formed by collapse settlement of poorly compacted fill
Fill layer

Figure 8.17 Crack or hydraulic fracture in poorly compacted layer in the embankment.

Table 8.7 Relative proportion of incidents for the different crack mechanisms (Fell *et al*., 2008, based on Foster and Fell, 1998, 2000a).

Cracking mechanisms		*No. of cases*	*% of cases*
Cracking mechanisms	Differential settlement, cross valley	20	35%
	Differential settlement, cross section	6	11%
	Differential settlement, foundation	4	7%
	Differential settlement, embankment staging	0	0%
	Desiccation cracking	3	5%
	Closure section	3	5%
	Total for cracking mechanisms	36	63%
Poorly Compacted and High permeability zones	Foundation contact	5	9%
	In embankment	16	28%
Total for Poorly Compacted and High Permeability Zones		21	37%

This has been caused by settlement of the soil in the saturated zone and arching of the unsaturated soil over the saturated material. In these cases the gap may be a pathway for water at reservoir levels greater than the historic high.

The large number of piping incidents experienced in Swedish central core earth and rock fill dams (Nilsson, 2007a, b) constructed with broadly graded glacial till cores may be initiated by collapse settlement of the core which was lightly compacted in 800 mm thick layers. Lawton *et al*. (1992) consider the collapse settlement issue.

Figure 8.18 shows concentrated leaks formed in Tunbridge Dam, Tasmania. Given the multiple leaks at the same level it appears likely that these formed by collapse settlement of a poorly compacted layer or on a construction shut down surface which was not properly compacted.

Table 8.8 Amount of collapse compression which may occur on saturation versus compaction properties (Fell *et al*., 2008).

Description of the method and degree of compaction of the core or soil around the conduit		*Amount of collapse compression as a proportion of the layer thickness*
Soil placed with no formal compaction (e.g. by horse and cart in old dams), or by pushing into place by excavator or bulldozer or in very thick layers. No control of layer thickness.	Layers very poorly compacted dry of standard optimum moisture content e.g. <90% standard dry density ratio, 3% dry of standard OMC	0.02 to 0.05
Well dry of optimum moisture content. Soil placed and compacted by bulldozer, no compaction by rollers, or rolled in thick layers beyond the capability of the roller. Layer thickness at or beyond the limit of compaction equipment. Dry of optimum moisture content.	Layers very poorly compacted dry of standard optimum moisture content e.g. <93% standard dry density ratio, 2% to 3% dry of standard OMC	0.01 to 0.02
Soil rolled in layers near the limit of the capability of the rollers, at moisture contents dry of standard OMC.	Compacted to e.g. 93%–95% standard dry density ratio, 2% to 3% dry of standard OMC	0.005
Soil compacted by suitable rollers in suitable layer thickness. Around optimum moisture content.	Well compacted to e.g. 95%–98% standard dry density ratio, 2% dry of standard OMC to 1% wet of standard OMC	Will not collapse
As above but with good documentation and records.	Very well compacted to e.g. >98% standard dry density ratio, 2% dry of standard OMC to 1% wet of standard OMC	Will not collapse



Figure 8.18 Concentrated leaks in Tunbridge dam possibly related to collapse settlement of a poorly compacted layer.

8.3.2.9 *Internal erosion associated with conduits embedded in the embankment*

Many internal erosion and piping failures and incidents occur where conduits are embedded in the embankment.

The conduit facilitates the initiation of internal erosion by:

- Causing stress distributions due to the stiff conduit and its less stiff surrounding soil which lead to low principal stresses and hydraulic fracture. This is discussed by Sherard *et al.* (1972a, b) and Charles (1997). These low stress zones can occur on the sides of culverts which are constructed in a trench. Sherard *et al.* (1972a, b) point out that they can also occur where the concrete culvert, or concrete surround around a pipe, has a sharp corner. In this case piping can be expected above the culvert.
- Drying of the soil in the sides of the trench in which the culvert is placed during construction can also cause cracks which allow initiation of piping.
- Making compaction of soil difficult, particularly if collars are provided at close intervals or the concrete is formed with corrugated steel sheet or other non-smooth formwork, preventing compaction of the soil adjacent to the conduit. It can also be difficult to compact the soil surrounding the conduit if the space between the conduit and the sides of the trench in which it is placed is narrow and compaction by rollers is not possible. Round pipe conduits that are not encased in concrete are especially susceptible to poor compaction of the soil beneath the pipe.
- Poor compaction is likely to lead to collapse settlement of the soil on saturation forming a gap adjacent to the conduit. Note that it is the continuity of the potential defect caused by a conduit and the excavation through the dam which is so critical to the initiation of internal erosion.

Figure 8.19 shows some of these features. Figure 8.20 shows the process of development of a concentrated leak along a conduit. In the figure the emphasis is on hydraulic fracture but it is just as likely that collapse settlwment of the poorly compacted soil beneath the pipe is the cause.

Internal erosion may also occur into open joints or other defects in conduits or into conduits which have deteriorated due to corrosion.

More information on conduits through embankments is provided in Section 13.5.

FEMA (2006) "Conduits through Embankment Dams. Best practices for design, construction, problem identification and evaluation, inspection, maintenance, renovation and repair" provides a comprehensive coverage of the issues relating to conduits in embankment dams.

8.3.2.10 *Cracking due to desiccation*

Desiccation cracking is most likely to be an issue in climates with less than 250 mm annual rainfall, in high plasticity cores, and where there is no surface layer over the core.

However based on a review of literature desiccation cracking can extend to significant depths. For example:

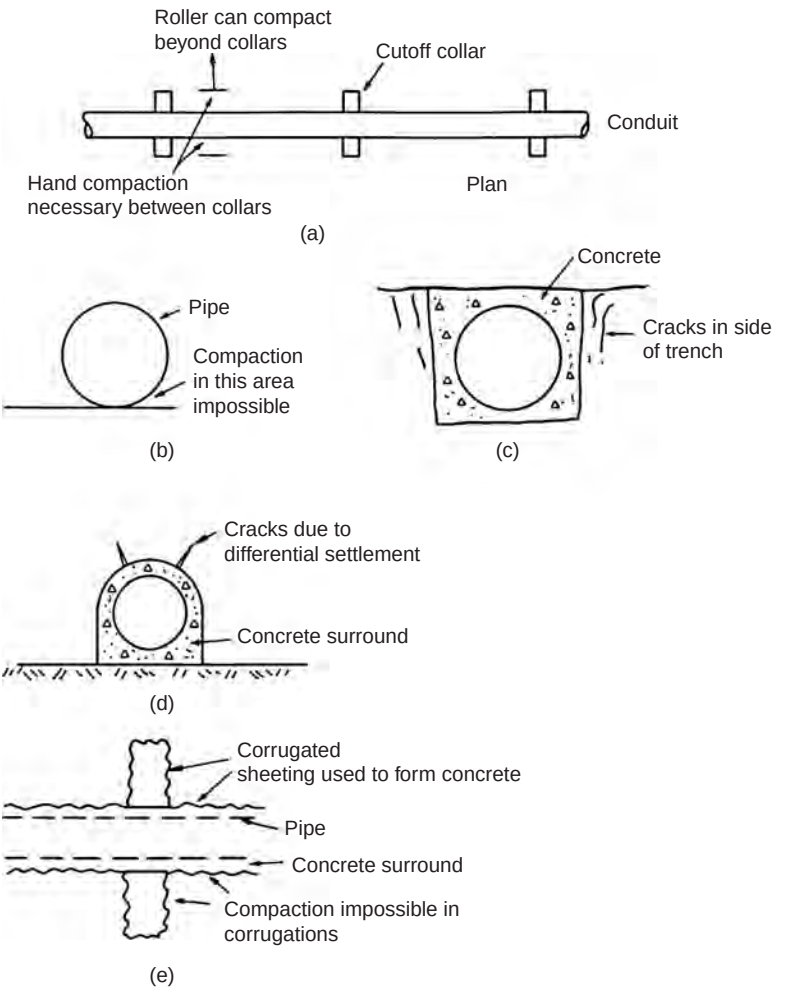
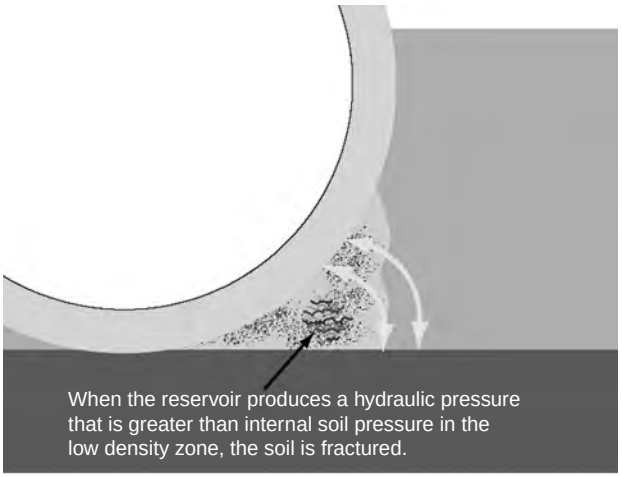


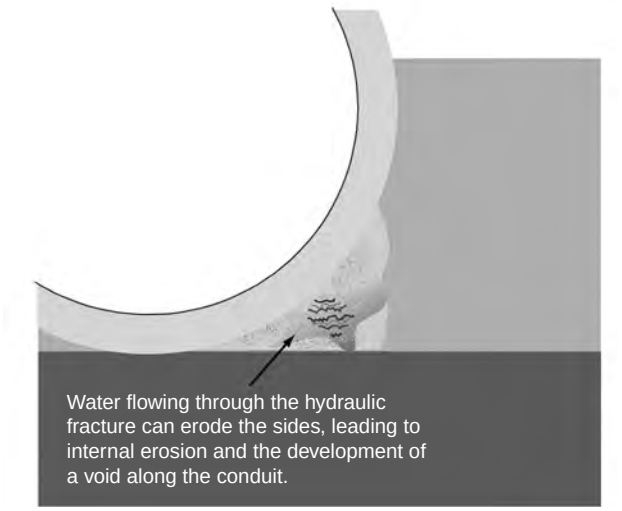
Figure 8.19 Some causes of internal erosion around conduits: (a) Inadequate compaction due to the presence of cutoff collars; (b) Inadequate compaction under pipe; (c) Cracking in soil or extremely weathered rock in the sides of a trench; (d) Cracks due to differential settlement; (e) Use of corrugations or other roughening of the surface of cutoff collars or concrete surround.

(i) Talbot (1994) reported desiccation cracking to a depth of 15 ft (5 m) in arid regions of the USA even in soils not associated with high shrinkage and swelling. These were spaced up to 7 m to 10 m and were a few millimetres wide

(ii) Lawrence (2002) summarizing the outcome of the "Workshop on cracking of homogeneous earth dams in semi-arid environments", indicated that a number of dams in Maricopa Country, Arizona, had experienced extensive desiccation cracking. Some details are:


When the reservoir produces a hydraulic pressure that is greater than internal soil pressure in the low density zone, the soil is fractured.

Poor compaction causes arching to occur in the area of the conduit's haunches. This creates a low density zone subject to hydraulic fracture.


Water flowing through the hydraulic fracture can erode the sides, leading to internal erosion and the development of a void along the conduit.

The hydraulic fracture can propagate in a downstream direction and initiate flowing water from the reservoir.

Figure 8.20a The process of development of a concentrated leak along a conduit die to poor compaction. Hydraulic fracture or collapse settlement on saturation causes a crack to initiate (FEMA, 2006).

- The 22 dams are up to 100 ft (32 m) high, but most are less than 40 ft (13 m high), with a combined crest length of 19 km. The dams are 26 to 48 years old.
- The average rainfall is 7 to 10 inches (175 mm to 250 mm), with an annual net evapo-transpiration of 68 to 76 inches (1700–1900 mm).



Water from the reservoir penetrates the hydraulic fracture, initiating internal erosion of the side walls.



Flowing water from the reservoir continues the internal erosion process within the hydraulic fracture. Intergranular seepage is not necessarily involved in this process, and the embankment materials surrounding the void may be unsaturated.

Figure 8.20b The process of development of a concentrated leak along a conduit due to poor compaction. Progression of the hydraulic fracture or crack formed by collapse settlement along the conduit FEMA (2006).

- The soils in the embankments classify as SM, SC, CC-ML, CL and ML, and were compacted to 100% standard maximum dry density (SMDD) at or near optimum water content.
- The soils had dried by about 6% to 7% since construction.
- Desiccation cracks extended to a depth of 20 ft (6.4 m), with an average spacing of 12 ft (3.6 m). The cracks were up to 75 mm wide.

(iii) Desiccation cracking has also been noted in a number of Australian dams. Most noticeably Ross River Dam, where cracks extended to a depth as great as 13.6 m and were the initiator for surface tunnel erosion and piping due to rainfall flowing in the cracks. The desiccation cracking which has been the result of many years of drying had a depth to width ratio of about 40 to 50.

(iv) Tandjiria *et al.* (2005) report on desiccation trials at three dam sites in Australia. These were used to validate a model for desiccation. They showed that the

Table 8.9 Maximum likely depth of desiccation cracking in dam cores depending on climate and crest road details (Fell *et al.*, 2008).

Climate	*Maximum likely depth with gravel layer with no road pavement cover[(a)] (metres)*	*Maximum likely depth with gravel layer with no road pavement cover[(a)] (metres)*
Arid climate, less than 10 inches (250 mm) rainfall, high summer temperatures	4.5	7
Monsoonal or other distinct wet and dry periods in the year. Summer maximum temperatures >85°F (>30°C)	4	6
Seasonal climate with annual rainfall greater than 20 inches (500 mm)	3	5
and no prolonged hot dry periods Temperate climate, uniform rainfall throughout the year	2	4.5

Note: [(a)] Road pavement cover may comprise concrete, asphalt or bitumen seal.

depth to width ratio for cracking increased with time from generally about 6 after 1 month to 12 and 15 after 2 and 3 months.

Based on these case data Fell *et al.* (2008) developed Table 8.9 which gives maximum likely depth of desiccation cracking. If the dam has significant freeboard desiccation cracking may only be reached by the reservoir under flood conditions.

Desiccation cracking may also occur on seasonal shut down surfaces during construction, or on the surface of the first stage of dams which are built in two stages. Good construction practise would be to remove the desiccated soil but this was not always done.

Experience in excavating into the crest of embankments is that if there is a road pavement of granular road base or a rockfill or other non-plastic layer at least 300 mm thick cracking is not generally observed even in seasonal climates with extended dry periods. The likelihood of cracking is further reduced if the road pavement is sealed with asphalt, concrete or bitumen seal. In such situations where cracking is observed it is narrow and does not persist to a significant depth.

This is to be expected because the equilibrium moisture content beneath paved surfaces or concrete slabs is around the standard optimum moisture content so if fills were compacted close to standard optimum moisture content, as most dam cores will be, there should little or no moisture change and as a result little or no cracking.

8.3.2.11 *Transverse cracking caused by settlement during earthquakes*

When embankment dams experience a large earthquake they often settle and spread in the upstream-downstream direction as shown in Figure 8.21. Many exhibit longitudinal cracking, and some transverse cracking. Some examples are Austrian Dam shown in Figure 8.22 (Harder *et al.*, 1991; Forster and McDonald, 1998;

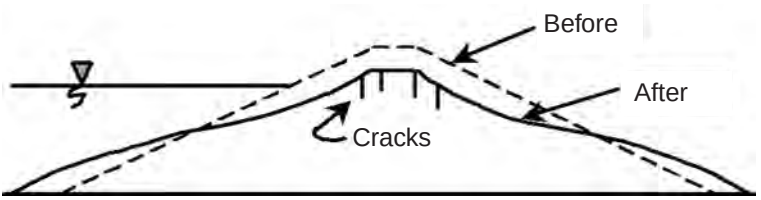


Figure 8.21 Settlement, lateral spreading and longitudinal cracking of embankment dams in earthquakes.

Vrymoed and Lam, 2006); Guadeloupe dam (Harder *et al.*, 1991) and Lexington dam (Fong and Bennett, 1995).

Pells and Fell (2002, 2003) analysed case data for dams not subject to liquefaction which showed that dams which settled more than about 1.5 % of their height were almost certain to exhibit transverse cracks; those which settled between 0.5% and 1.5% had about a 20% chance of exhibiting transverse cracks; and dams which settled between 0.2% and 0.5% had about a 5% chance of transverse cracking. Details of these data are provided in Section 12.6.6.2.

These deformations may be estimated by numerical analyses methods or the empirical data base methods described in Section 12.7.6.

These cracks are in the upper part of the embankment and whether they are a potential initiator of internal erosion depends on the reservoir level and depth of cracking.

It should be noted that Austrian dam only settled about 0.5% but developed wide deep transverse cracks near the spillway wall as shown in Figure 8.22.

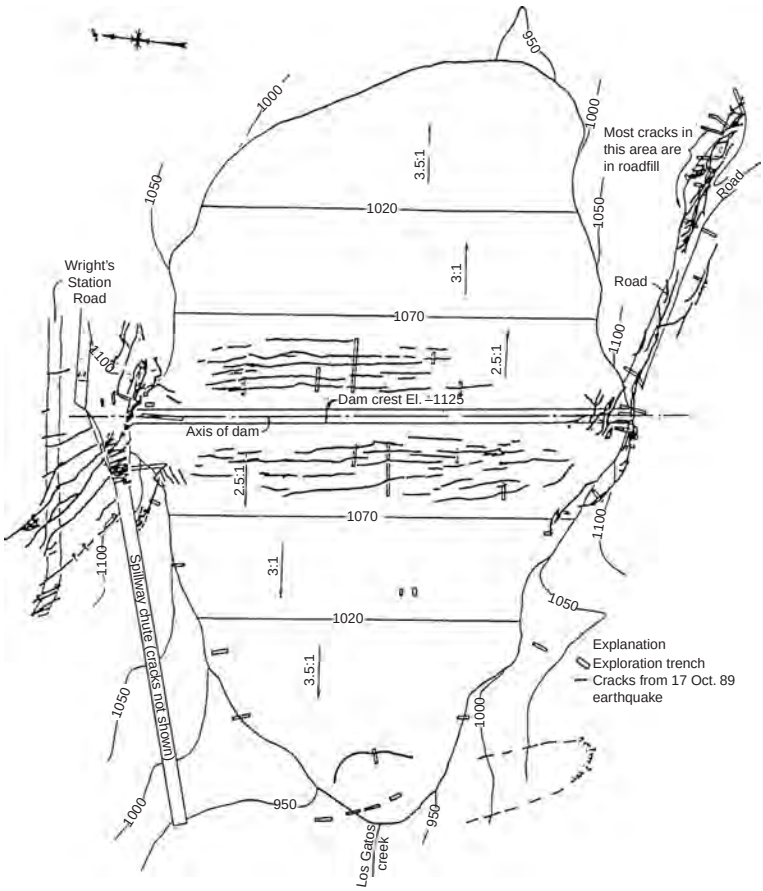
If liquefaction occurs the deformations are likely to be large and the likelihood of cracking greater.

8.3.2.12 *Cracking or high permeability layers due to freezing*

The effects of frost on embankment dams are described in Vuola *et al.* (2007). The effects of frost are to cause extra water to be drawn into the soil by capillarity action. This causes ice lenses to form with associated heave and potentially cracking and/or loosening of the soil. The cracking is most likely to be longitudinal but may also be transverse at the crest of the dam. Figures 8.23, 8.24 and 8.25 show the potential effects.

Vuola *et al.* (2007) provide an equation for estimating the depth of frost penetration.

There are a number of references which provide information on the susceptibility of soils to frost penetration. These include Wallace (1987), Holtz and Kovacs (1981), Vuola *et al.* (2007) and USACE (1984) which gives the most specific data. Core materials which are most susceptible to freezing and ice lens formation include silts, clayey silts, silty sands, silty gravels and clayey sands and gravels with a plasticity index <12.



(a)



(b)

Figure 8.22 Transverse and longitudinal cracking of Austrian Dam during the Loma Prieta earthquake: (a) Plan of the cracking. (b) Cracking adjacent to the spillway. (Forster and McDonald, 1998, Photograph courtesy of S. Todaro).

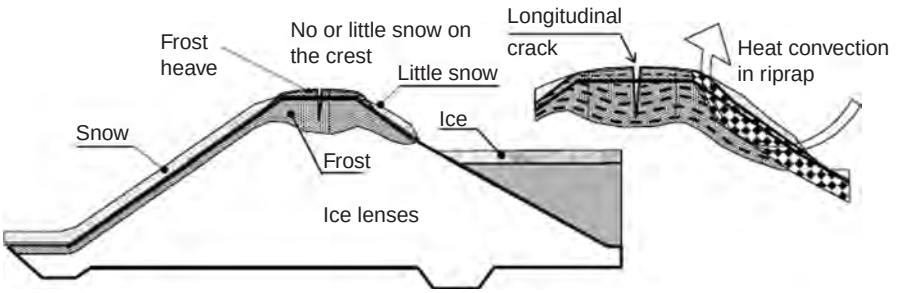


Figure 8.23 Effects of frost on an embankment dam (Vuola *et al.*, 2007).

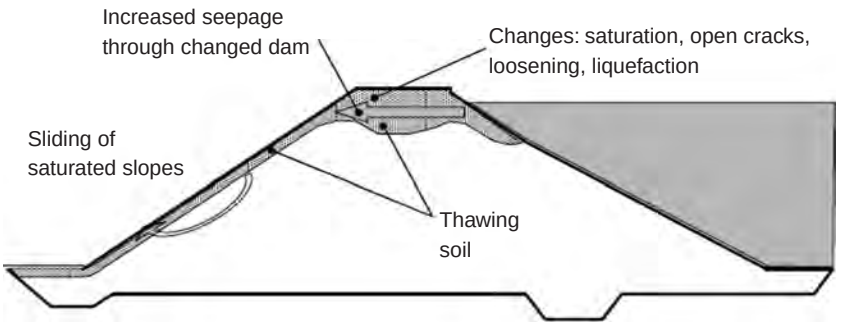


Figure 8.24 Damage phenomena occurring in embankment dams from frost thaw (Vuola *et al.*, 2007).

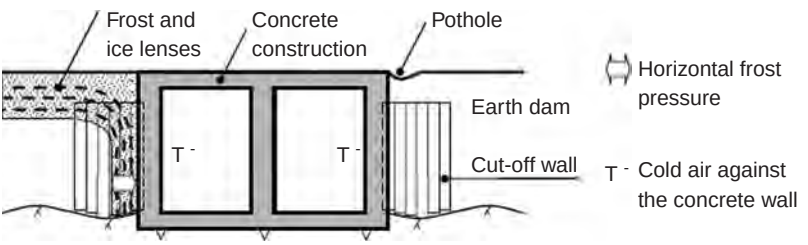


Figure 8.25 Effect of frost against a spillway structure (Vuola *et al.*, 2007).

8.3.2.13 *Internal erosion initiated by the effects animal burrows and vegetation*

Animal burrows in an embankment or levee can lead to a situation where there are nearly continuous holes through the embankment or situations like those shown in Figure 8.26 where high gradients between holes may result in initiation of erosion.

FEMA (2005a) "Impacts of animals on Earthen Dams, FEMA Report 473" provides a comprehensive coverage of the issues.

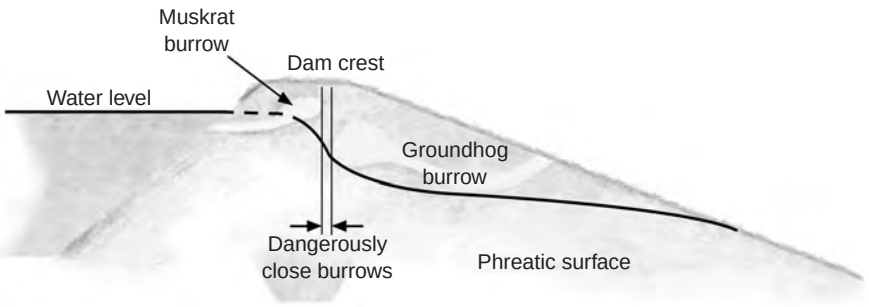


Figure 8.26 Potential impacts of burrowing animals on internal erosion in embankments and levees (FEMA, 2005a).

Vegetation growing on dams and levees can cause situations which may lead to a potential concentrated leak and initiation of erosion. The effects include:

- Decaying roots that create seepage paths. This is a serious issue and the authors are aware of a number of piping incidents related to root in embankment dams.
- Roots penetrating into open joint and cracks in foundation rock, potentially creating seepage paths.
- Root penetration of conduit joints and joints in concrete structures and opening the joints to allow erosion into or out of the conduit or wall.
- Clogging embankment under-drain systems.
- Interfering with effective visual inspection.

FEMA (2005b) Technical Manual for Dam Owners, Impacts of Plants on Earthen dams, FEMA Report 534 provides information and guidance on how to manage this problem.

8.3.2.14 *Relative importance of conduits, spillway walls cracking mechanisms, and poorly compacted zones*

As detailed in Table 8.2, about half of internal erosion incidents in embankments have historically involved conduits through the embankment and spillway walls supporting the embankment. These are related to poor compaction, differential settlement and arching effects.

Excluding conduits and spillway wall the relative proportion of incidents in the Foster *et al.* (1998, 2000a) ICOLD database by cracking and poorly compacted zones is:

- Differential settlement cracking – 36/57 = 63%.
- Poorly compacted and high permeability zones – 21/57 = 37%.

A further assessment of the case study information results in Table 8.7.

8.3.3 Estimation of crack width and depth of cracking

8.3.3.1 *Cracking due to differential settlement, adjacent walls*

The width and depth of cracks which may be present in the embankment may be estimated using methods detailed in Fell *et al.* (2008). These methods are reproduced in Appendix A.

These are based on a review of the literature on observed cracking including Sherard (1973), Talbot (1994) and Lawrence (2002), and the results of numerical modelling by Bui *et al.* (2004, 2005) which assists in assessing the likely depth of cracking.

Sherard (1973) describes cracks up to 75 mm wide, and widths from 3 mm to 20 mm as common. Transverse cracking at Buffalo dam (Newman and Foster, 2006) was up to 40 mm wide, and diagonal cracking on the downstream slope of Tullaroop dam (Hunter *et al.*, 2007) was up to 60 mm wide. The cracks become narrower at depth and, for practical purposes, it may be assumed that they have a linear taper from a maximum at the surface to zero at maximum depth. The evidence is this may be on the conservative side with cracks generally narrowing more rapidly with depth. Based on Sherard *et al.* (1972a, b) it seems unlikely cracks due to hydraulic fracture will exceed 5 mm to 10 mm width but this is difficult to predict.

The methods for estimating crack widths in Fell *et al.* (2008) and Appendix A are based on these data on maximum crack widths; relating the potential maximum crack width at the crest of the dam being assessed to the likelihood of cracking being present and allowing for a judged distribution of likely crack widths.

The values estimated by these methods are very approximate and this uncertainty should be allowed for in assessing internal erosion.

The preferred method is to do a numerical model with the profile of the valley of the dam in question, using Cam-Clay or Mohr-Coulomb models.

From this analysis:

- Estimate potential depths of low stress zones from cross valley numerical analysis.
- Determine cumulative extensional strains from these analyses and from this estimate the upper bound width of cracking assuming all strain concentrates on a single crack.
- Make some judgment on the likely crack width using this value and the data in Appendix A.

This should be done for important projects and is relatively routine. 2D analyses are generally adequate. However if the dam is a central core dam with dumped (not compacted) rockfill large lateral deformations will occur and these increase settlements and the likelihood of transverse cracking. For these cases 3D analyses are preferred but are more expensive to do.

It is currently not possible to model numerically the potential width of cracks accurately because that requires knowledge of crack spacing as well as strains. Research is under way at UNSW to develop improved methods for numerically modelling the depth and width of cracking.

8.3.3.2 *Cracks formed by collapse settlement of poorly compacted soil*

For collapse settlement of a poorly compacted layer it is most practical to assume a gap or flaw is formed and to assess the likelihood of erosion initiating in the flaw. The procedure suggested by Fell *et al.* (2008) to estimate the height of the gap or flaw is:

- Assess the thickness of the layer of soil which is poorly compacted (T_p). There may be a single layer or several layers.
- Estimate the amount by which the layer may collapse (C_F) using Table 8.8. Then estimate the height of the gap (flaw) which could result (G) from $G = (T_p) \times (C_F)$. This represents the scenario with the weight of the soil above being supported on the adjacent non-collapsed soil.

This method should be also applied to silty sands, and silty sandy gravel soils which may be subject to collapse settlement even if the soils are non plastic, since they will erode rapidly in a gap.

For poorly compacted soil around a conduit the suggested procedure is:

- Assess the thickness of the layer of soil which is poorly compacted. (T_p). There may be a single layer or several layers.
- Estimate the amount by which the layer may collapse (C_F) using Table 8.8 as a guide. Then estimate the height of the gap (flaw) (G) which could result from $G = (T_p) \times (C_F)$. For conduits placed in trenches the dimensions of the conduit and the trench in which it is placed must be taken into account in this estimate.
- For cases where it appears that the soil around the pipe is well compacted and where gap widths <5 mm are calculated, assume a gap width of 5 mm to allow for possible shrinkage of the soil from the pipe during construction or in service.

8.3.4 The mechanics of erosion in concentrated leaks

8.3.4.1 *The procedure for assessing whether erosion will initiate*

The procedure for assessing whether erosion will initiate in a crack or flaw is to:

- Estimate the hydraulic shear stresses in the crack for the reservoir level under consideration, taking account of the geometry of the core of the embankment and the assumed crack dimensions and location relative to the reservoir surface so the flow gradient can be determined.
- Compare this hydraulic shear stress to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c) at the degree of saturation of the soil on the sides of the crack. In doing this take account of the dispersion properties of the soil and the chemistry of the seepage water.

There is always some uncertainty regarding the input parameters so the analysis should check the sensitivity to the assumptions made.

The following sections give an outline of this process.

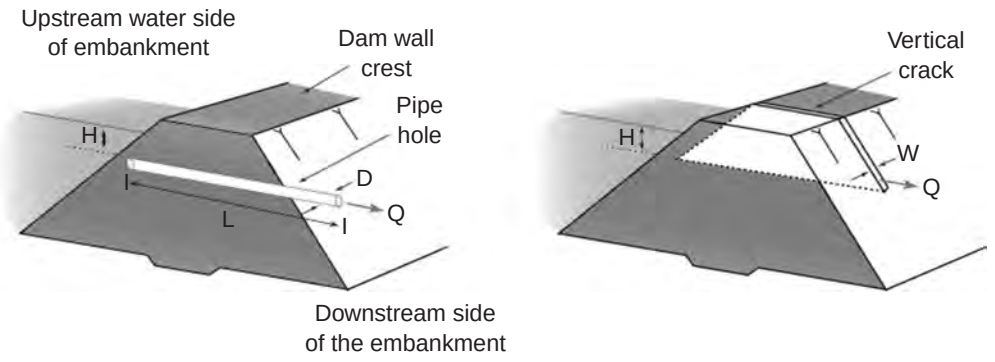


Figure 8.27 Geometry of pipes and cracks for estimation of hydraulic shear stresses.

8.3.4.2 *The estimation of hydraulic shear stresses in cracks and pipes*

Wan (2006), Wan and Fell (2002, 2004a, b) and Fell *et al.* (2008) give details on how to estimate the hydraulic shear stresses in cracks and cylinders as shown in Figure 8.27.

They give the following equations for estimating the hydraulic shear stress on the surface of a cylindrical pipe, or parallel sided transverse crack in an embankment. The assumptions are:

- Linear head loss from upstream to downstream
- Steady uniform flow along the crack
- Zero pressure head at the downstream end
- Uniform frictional resistance along the surface of the crack or cylindrical pipe
- Driving force = frictional resistance.

(a) Cylindrical pipe:

$$\tau = \rho_w \frac{gH_f d}{4L} \tag{8.1}$$

(b) Vertical transverse crack

$$\tau = \frac{\rho_w \, gH_f^2 W}{2(H_f + W)L} \tag{8.2}$$

where τ = Hydraulic shear stress in N/m^2
ρ_w = Density of water in kg/m^3
g = Acceleration due to gravity = 9.8 m/s^2
H_f = Head loss in pipe or crack due to friction in metres
L = Length of pipe or crack base in metres
D = Diameter of the pipe in metres
W = Width of crack in metres

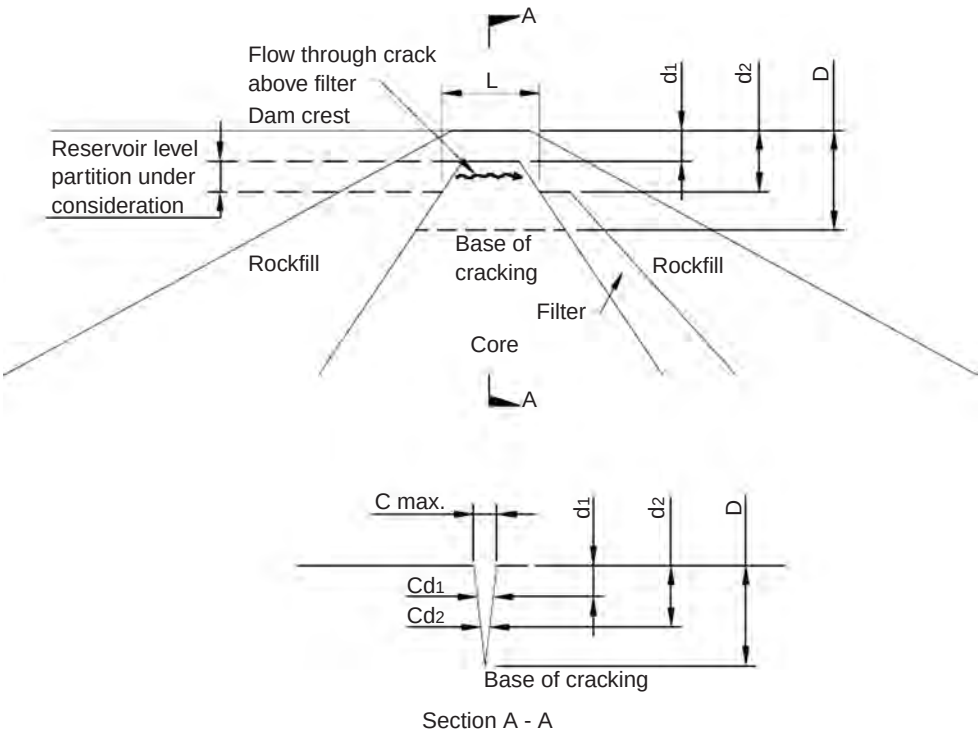


Figure 8.28 Detailed geometry of vertical crack in the top of the core for estimation of the hydraulic shear stress in the crack (Fell *et al.*, 2008).

In practice the geometry of the core and the width of the crack vary as shown in Figure 8.28 and this should be allowed for in the calculations. This can be done by:

- Estimate likely crack width at the reservoir level stage under consideration. The width required is the width at the mid level of the flow path for the reservoir stage under consideration. In Figure 8.28 it is the average of C_{d1} and C_{d2}.

  The width C_{d1} at depth $d_1 = C_{max}((D - d_1)/D)$. That is, the crack is assumed to be uniformly tapered from the dam crest to the base of the crack.
- Estimate the gradient of flow through the crack. The gradient required is the average gradient at the mid level of the flow path for the reservoir stage under consideration. In Figure 8.28 it is $(d_2 - d_1)/L$.

8.3.4.3 *Erosion properties of soils in the core of embankment dams – basic principles*

The resistance to initiation of erosion of the core to concentrated leak erosion is characterized by the critical shear stress and the rate of enlargement of the pipe in the progression phase is characterized by the erosion coefficient.

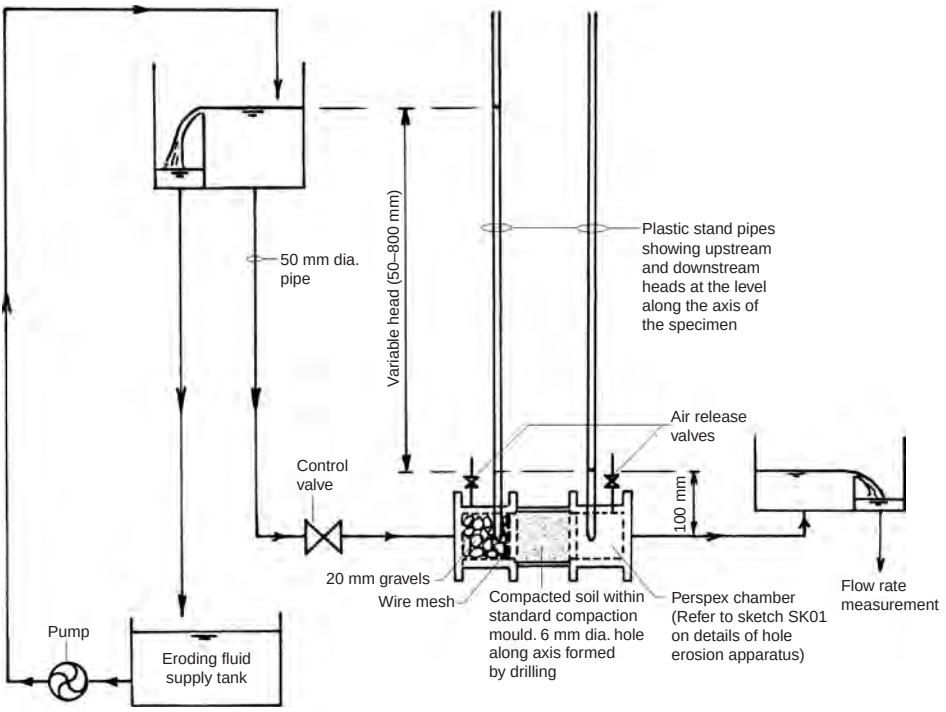

50 mm dia. pipe
Variable head (50–800 mm)
Plastic stand pipes showing upstream and downstream heads at the level along the axis of the specimen
Air release valves
Control valve
100 mm
20 mm gravels
Wire mesh
Compacted soil within standard compaction mould. 6 mm dia. hole along axis formed by drilling
Perspex chamber (Refer to sketch SK01 on details of hole erosion apparatus)
Flow rate measurement
Pump
Eroding fluid supply tank

Figure 8.29 Hole Erosion Test equipment (Wan and Fell, 2002, 2004a, b).

The erosion properties of soils for concentrated leak erosion can be determined by rotating cylinder tests e.g. Arulanandan and Perry (1983); Chapius (1986a, b); Chapius and Gatien (1986); Lim (2006); Lim and Khalili (2010); Slot Erosion Tests (SET) and the Hole Erosion Tests (HET) e.g. Wan (2006); Wan and Fell (2002, 2004a, b); Bonelli *et al.* (2007) and Jet Erosion tests, e.g. Hanson (1990, 1991); Hanson and Cook (2004). These tests allow determination of the critical shear stress for initiation of erosion, and the erosion rate. At this time the most widely used tests are the Hole Erosion Test and the Jet Erosion Test. Figure 8.29 shows the laboratory test set up for the HET. Figure 8.30 shows a plot of rate of erosion ($\dot{\varepsilon}$) versus hydraulic shear stress (τ) showing extrapolation to determine the critical shear stress (τ_c) at zero rate of erosion and the coefficient of soil erosion (C_e). In practice the data is seldom of such good quality as shown in Figure 8.30 and Wan and Fell (2002, 2004a, b) prefer to determine the critical shear stress by varying the gradient in the test. They called this the initial shear stress.

Wan and Fell (2002, 2004a, b) and Wan (2006) expressed the erosion rate in the form of an Erosion Rate Index, I_{HET} defined by:

$$I_{HET} = -\log(C_e) \tag{8.3}$$

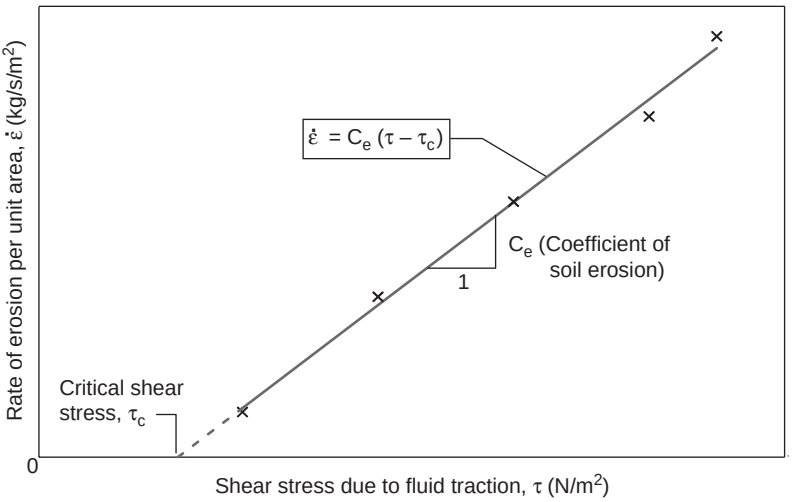

Rate of erosion per unit area, $\dot{\varepsilon}$ (kg/s/m^2)
$\dot{\varepsilon} = C_e (\tau - \tau_c)$
C_e (Coefficient of soil erosion)
1
Critical shear stress, τ_c
0
Shear stress due to fluid traction, τ (N/m^2)

Figure 8.30 Plot of rate of erosion ($\dot{\varepsilon}$) versus hydraulic shear stress (τ) showing extrapolation to determine critical shear stress (τ_c) at zero rate of erosion and coefficient of soil erosion (C_e).

Table 8.10 Descriptors for erosion rates of soils (Wan, 2006).

Group No.	*Erosion Rate Index*	*Description*
1	<2	Extremely rapid
2	2–3	Very rapid
3	3–4	Moderately rapid
4	4–5	Moderately slow
5	5–6	Very slow
6	>6	Extremely slow

The representative erosion rate index $\tilde{I}_{HET}$ is the hole erosion index I_{HET} for soil compacted to a density ratio of 95% of standard maximum dry density at optimum moisture content.

Soils can be classified into 6 groups according to their Representative Erosion Rate Index, $\tilde{I}_{HET}$. The 6 groups are as shown in Table 8.10. Note that this is a logarithmic scale and the rate of erosion of soils varies by up to five orders of magnitude.

In the absence of laboratory test values, the Representative Erosion Rate Index ($\tilde{I}_{HET}$) can be related approximately to soil properties. Table 8.11 has been developed from test data to give a first approximation to the likely range of $\tilde{I}_{HET}$ for different classifications of non-dispersive soils.

The critical shear stress is related to the Erosion Rate Index as shown in Figure 8.31. The approximate estimates and likely range of critical shear stress (τ_c) in Table 8.12 may be used, with caution, when Hole Erosion Test values are not available.

Table 8.11 Representative erosion rate index ($\tilde{I}_{HET}$) versus soil classification for non dispersive soils based on Wan and Fell (2002).

	Erosion Rate Index ($\tilde{I}_{HET}$)		
Unified Soil Classification	*Likely Minimum*	*Best Estimate*	*Likely Maximum*
SM with <30% fines	1	<2	2.5
SM with >30% fines	<2	2 to 3	3.5
SC with <30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit <65%	3	4	5
CH with Liquid Limit >65%	4	5	6

Notes. (1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.

It is emphasised that it is better to carry out a series of Hole Erosion Tests at varying heads or to use the method of Bonelli *et al.* (2007); Bonelli and Brivois (2008) to define the critical shear stress (τ_c) than to rely on these relationships.

Three other points to note are the effect of degree of saturation of the soil and the effect of dispersion.

8.3.4.4 *Effect of degree of saturation of the soil*

The discussion in Section 8.3.4.3 relates to the properties of soils at the representative compaction conditions. That is at 95% standard maximum dry density and standard optimum moisture content. Wan and Fell (2002, 2004a, b) and Lim (2006), have found that the Erosion Rate Index for the Hole Erosion (I_{HET}) and Rotating Cylinder (I_{RCT}) tests are related to the degree of saturation of the soil.

Figure 8.32 shows results of testing three soils. Soil S is a residual granitic soil from Serpentine Dam, Western Australia classifying as MH, Soil B is a sandy clay of low plasticity from Fairbairn Dam, Queensland, Soil F is a clay of low plasticity from Boggy Creek Dam in Oklahoma, USA, and 50% of kaolin is an artificial soil consisting of 50% kaolin and the remainder fine sand size particles.

It will be seen that there is a significant dependence of the erosion rate on degree of saturation for the two clay soils. There was however less dependence on the degree of saturation for the silty soil. Wan and Fell (2002) noted a similar trend with two other residual granitic soils showing little change in the erosion rate with increased degree of saturation.

Lim (2006) noted that there was little change in the erosion rate for clay soils for degrees of saturation above 90%. Clay soils Lim tested which had $I_{RCT} \approx 3$ to 4 at the representative compaction and moisture content had $I_{RCT} \approx 4.5$ to 5.5 when 90% to 100% saturated. This is an important finding because it means that once the core of

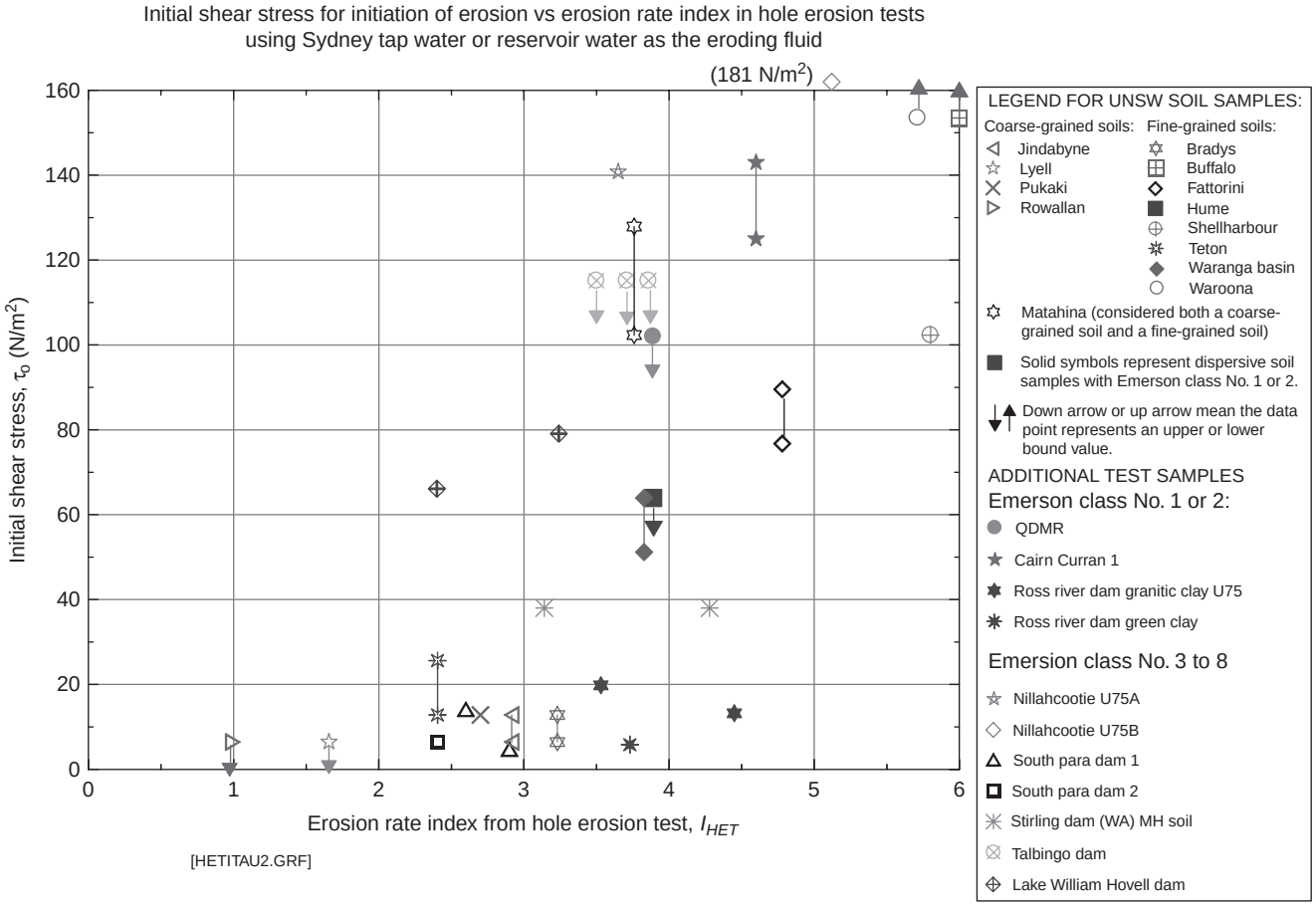

Initial shear stress for initiation of erosion vs erosion rate index in hole erosion tests
using Sydney tap water or reservoir water as the eroding fluid
(181 N/m2)
Initial shear stress, τo (N/m2)
Erosion rate index from hole erosion test, IHET
[HETITAU2.GRF]
LEGEND FOR UNSW SOIL SAMPLES:
Coarse-grained soils:
Jindabyne
Lyell
Pukaki
Rowallan
Fine-grained soils:
Bradys
Buffalo
Fattorini
Hume
Shellharbour
Teton
Waranga basin
Waroona
Matahina (considered both a coarse-grained soil and a fine-grained soil)
Solid symbols represent dispersive soil samples with Emerson class No. 1 or 2.
Down arrow or up arrow mean the data point represents an upper or lower bound value.
ADDITIONAL TEST SAMPLES
Emerson class No. 1 or 2:
QDMR
Cairn Curran 1
Ross river dam granitic clay U75
Ross river dam green clay
Emersion class No. 3 to 8
Nillahcootie U75A
Nillahcootie U75B
South para dam 1
South para dam 2
Stirling dam (WA) MH soil
Talbingo dam
Lake William Hovell dam

Figure 8.31 Critical (initial) shear stress (τ_o) versus representative erosion rate index ($\tilde{I}_{HET}$) for soils which are non-dispersive, and for dispersive soils with eroding water suppressing dispersion (Courtesy of C.F. Wan).

Table 8.12 Approximate estimates and likely range of initial shear stress (τ_{Co}) versus Hole Erosion Index (I_{HET}) (Fell *et al.*, 2008).

	Critical Shear Stress (τ_c) Pa			
Hole Erosion Index (I_{HET})	*Non Dispersive Soil Behaviour*		*Dispersive Soil Behaviour*	
	Best Estimate	*Likely Range*	*Best Estimate*	*Likely Range*
<2	2	1 to 5	1	0.5 to 2
2 to 3	2	1 to 5	1	0.5 to 2
3.5	5	2 to 20	2	1 to 5
4	25	10 to 50	5	2 to 10
5	60	25 to 100	5	2 to 10
6	100	60 to 140	5	2 to 10

Note. To be used with caution. For important decisions carry out Hole Erosion Tests to determine the critical shear stress (τ_c).

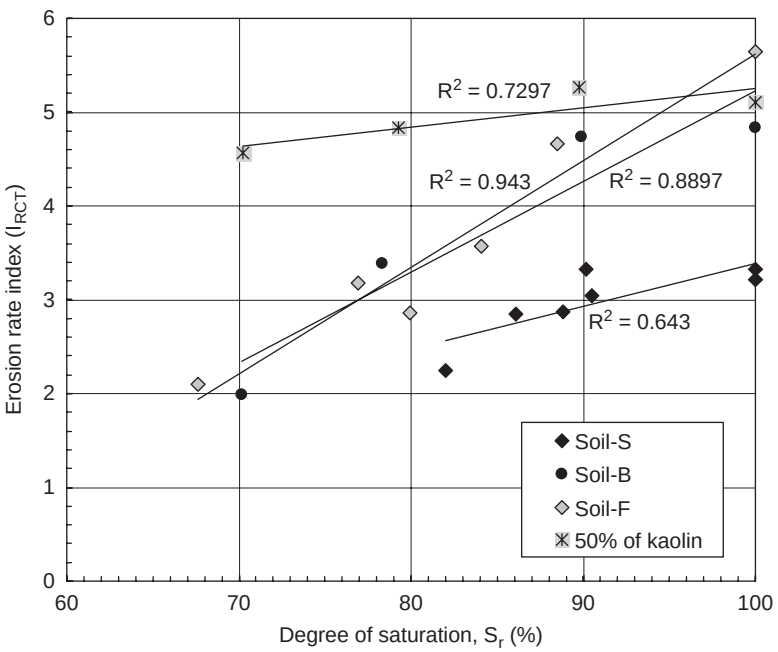


Figure 8.32 Relationship between degree of saturation and Erosion Rate Index (I_{RCT}) for non-dispersive soils (Lim, 2006).

a dam constructed of clay soil is saturated, it will have a slower rate of erosion, and a higher initial shear stress. Just as important is this does not apply to silty sand cores such as decomposed and residual granites.

From a practical point of view, it is therefore better to compact cohesive soils to the normal requirements for dam cores; e.g. to 98% standard maximum dry density,

on the wet side of optimum, because the erosion resistance increases with the degree of saturation of the soil.

8.3.4.5 *Effect of the testing method on the critical shear stress to initiate erosion (τ_c) and the erosion rate index*

Critical shear stress

The critical shear stress to initiate erosion (τ_c); (or for the HET the initial shear stress, τ_0) is a difficult property to measure. Lim (2006), Lim and Khalili (2010) have found the form of the hydraulic shear stress versus erosion rate plot varies for different soils, with some plotting as bi-linear plots, others with the intercept on the "X" and "Y" axes. Wan and Fell (2002) had similar problems and this is the reason they adopted the procedure of varying heads in the HET to define the initial shear stress.

Bonelli and colleagues at CEMAGREF have refined the HET and developed an improved method for determination of the critical shear stress. Details are given in Bonelli *et al.* (2007), Bonelli and Brivois (2008), and Benahmed and Bonelli (2012).

The roughness of the eroding surface affects the critical shear stress in both the HET and JET. In the HET the critical shear stress using the Bonelli *et al.* approach is measured at the end of the test so this effect is overcome. In the Wan method the early part of the test has to be ignored or this effect will control the results. In the JET different results are obtained if the upper and lower surfaces of the sample are tested reflecting the differences in roughness of the surface.

Erosion rate index

Wan and Fell (2002, 2004a, b) obtained a strong correlation between the erosion rate index from the Hole Erosion Test and the Slot Erosion Test. In the Hole Erosion Test erosion is from the sides of an initially 6 mm diameter hole drilled into a 100 mm long by 100 mm diameter cylindrical sample oriented horizontally. In the Slot Erosion Test erosion is from the sides of an initially 10 mm by 2.2 mm cross section by 1 metre long slot oriented horizontally.

Lim (2006), Lim and Khalili (2010) investigated the erosion properties of soils using rotating cylinder and Hole Erosion Tests. In the rotating cylinder test the erosion is from the vertical sides of a 100 mm diameter by 100 mm high sample. Figure 8.33 shows a comparison between the erosion rate indices for these tests. It can be seen that for dispersive soils there is good correlation with the HET giving slightly larger index (slower rate of erosion). However there is a large difference for the non-dispersive soils with the rotating cylinder test giving rates 10 to 15 times those in the HET. Lim (2006) attributes this to the vertical face of the RCT, and to surface and/or body slaking occurring in the RCT which could not occur in the HET because of the relatively large size of the slaking particles and the small hole in the HET.

Wahl *et al.* (2008) carried out tests where identically prepared remoulded soil samples were tested by the Hole Erosion and Jet Erosion Tests. They found that both methods ranked the soils similar for their relative erodibility, but the JET method indicated a higher rate of erosion up to one or more orders of magnitude, and lower critical shear stresses by up to two or more orders of magnitude. The speculated that

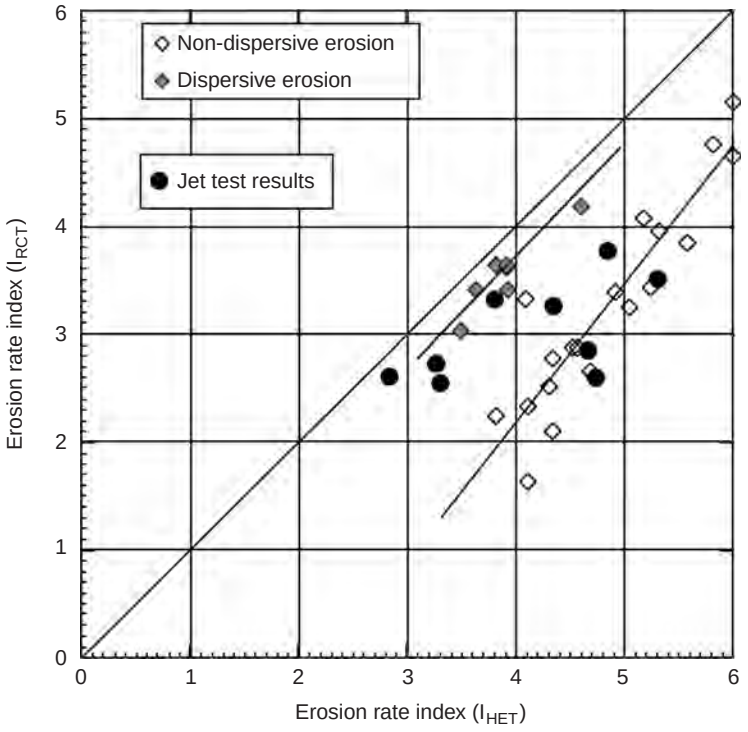


Figure 8.33 Correlation between erosion rate index from Rotating Cylinder and Hole Erosion Tests by Lim (2006), with the results of and Jet Erosion Tests by Wahl *et al.* (2008) superimposed.

differences between the erosion rates observed in the two tests were related to several factors, including:

- Differences in the nature of the hydraulic attack upon the eroding surface in each test
- Differences in the way that the flow exploits different weaknesses in the soil structure
- Differences in the geometry of the exposed soil surface
- Inaccurate or incomplete analytical models for the stress distributions produced at the soil surface
- Use of a linear erosion model for analysis, when erosion rate versus applied stress may actually be nonlinear
- Performing tests in different stress ranges and with an opposite progression of applied stress (high to low in JET, and typically well above the critical stress; low to high in HET, beginning near the critical stress level).

The range of soil types and compaction conditions for the tests by Wahl *et al.* (2008) was limited, but the greatest differences seemed to occur in samples with a

coarse or non-uniform soil structure (generally samples containing more clay and compacted significantly dry of optimum water content). This suggested that the JET is more sensitive to soil fabric, perhaps because the stress environment produced by the impinging jet is more readily able to exploit weak zones in a non-uniform soil structure. This may also be related to the different geometry of the exposed soil surface in each test: a planar surface in the JET versus a small, confined hole for the HET. Aggregates of soil particles that can be dislodged and removed in the JET environment may be held in place by the surrounding soil mass in the HET.

It is considered that the HET and SET better model erosion conditions in a crack and pipe in the early stages of development than does the rotating cylinder. However it could be that the HET and SET underestimate the rate of erosion as the pipe develops and the size allows the mechanisms modelled in the rotating cylinder to take effect. This could be allowed for by using a varying erosion rate index when calculating the rate of enlargement of a pipe.

8.3.4.6 *Effect of dispersion, slaking, soil structure and shear strength on erosion properties*

Dispersion

As discussed above, soils which show dispersive behaviour; soils classifying as Emerson Crumb Class 1 or 2 and Pinhole Dispersion D1 and D2, will have a very low critical shear stress if the eroding fluid does not inhibit the dispersion. That is if the eroding water has low salts content.

However, Lim (2006) showed that for rotating cylinder tests the Erosion Rate Index is not greatly affected by whether the soil is dispersive after the initially rapid part of the erosion process.

Slaking

The term "slaking" or "soil slaking" is defined as "*... disintegration of unconfined soil after exposure to the air and subsequent immersion in a fluid, usually water; no external confining pressure is assumed to act over the soil prior to immersion ...*" (Moriwaki and Mitchell, 1977).

Lim (2006) showed that slaking is the most important process involved in erosion in the RCT. They proved a strong correlation between the rates of slaking from a sample held statically in water to the erosion rate index from the RCT (I_{RCT}). He also showed that the slaking process was correlated strongly to the degree of saturation of the soil, with the slaking rate being up to 30 to 50 times lower between soils at 70% degree of saturation and those at 100% degree of saturation. This corresponds with the behaviour of the erosion rate index for clay soils.

This is a very important finding as it helps explain the actual mechanics of the erosion process in clay soils being strongly linked to slaking.

Soil structure

Benahmed and Bonelli (2012), Lim (2006) and Wahl *et al.* (2008) have all noted that soil structure has an important effect on the erosion properties. They find that erosion rates are significantly higher for the same soil if the soil is compacted dry of optimum

moisture content and the soil forms aggregated particles, and/or micro-cracks. These allow erosion of blocks of the soil rather than of individual particles. This is one of the reasons higher erosion rates are measured in JET than HET, as the HET test is stopped with a relatively small hole diameter not allowing the "blocks" of soil to dislodge from the sides of the hole. This behaviour was also noted in rotating cylinder tests by Lim (2006).

Shear strength of the soil

Contrary to what many have assumed, the erosion rate and critical shear stress are unrelated to the effective stress or undrained shear strength of the soil. Soils with similar undrained strengths can have an erosion rate 100 or 1000 times different and critical shear stresses ranging over 1 Pa to 100 Pa depending on dispersion properties, clay mineralogy and degree of saturation.

8.3.5 Comparison of the hydraulic shear stress in the crack (τ) to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c)

To assess the likelihood erosion will initiate in a crack, the estimated hydraulic shear stress is compared to the initial shear stress for the soil in the core taking account of the moisture content of the soil in the core, and the chemistry of the water in the reservoir. Table 8.13 is useful to assist in doing the assessment. It is based on the equations in Section 8.3.4.2. When doing this assessment allowance should be made for the uncertainty in the calculations and properties.

It should be noted that under flood conditions the salts content of the water in the reservoir is likely to reduce, so tests done in reservoir water may be un-conservative. If in doubt with dispersive soils it is best to assume the reservoir water will not inhibit dispersion and rely on the results of tests using distilled water.

It is important to recognise that erosion in cracks or flaws is not a result of establishment of a flow net, but is due to flow in open cracks. This occurs quickly as the reservoir rises into the cracked zone. Transient flow net analyses are irrelevant to this process.

Table 8.13 Estimated hydraulic shear stress (N/m^2) from water flowing in an open crack, versus crack width and flow gradient (Fell *et al.*, 2004).

	Flow Gradient in Crack					
Crack Width Millimetres	*0.1*	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.5	1.25	2.5	5	10	25
2	1	2.5	5	10	20	50
5	2.5	6	12	25	50	125
10	5	12	25	50	100	250
20	10	25	50	100	200	500
50	25	60	125	250	500	1250
100	50	125	250	500	1000	2500

8.4 BACKWARD EROSION

8.4.1 General description of backward erosion

There are two kinds of backward erosion.

Backward erosion piping. The erosion process begins at a free surface on the downstream side of a dam or dyke as shown in Figure 8.34. This free surface may be in a ditch or other excavation penetrating into the eroding cohesionless soil or may form by first heaving of the cohesive strata overlying the cohesionless soil.

Backward erosion piping initiates where critically high hydraulic gradients at the toe of a dam erodes particles upwards or erodes particles horizontally in the case where the cohesionless soil layer daylights in the side of a ditch.

The process progresses beneath the dike or dam in small erosion channels in which the flow velocity can transport the eroded particles downstream. For this to occur, the dike or dam, or the cohesive strata must form a roof for the eroding "pipe". The presence of backward erosion piping is often exhibited by the presence of sand boils at the downstream side of the dam or dike. Figure 8.35 shows an example of a sand boil.

The experience in USA and Europe is that backward erosion piping mostly occurs in the foundations of levees, dikes and dams where the eroding soil is fine to medium grain size sand, with a uniformity coefficient $Cu < 3$.

The participants at the Aussois Workshop (Fell and Fry, 2007) considered that at gradients likely to occur within a dam or its foundation backward erosion piping is probably restricted to cohesionless (non-plastic) soils or soils with only limited plasticity.

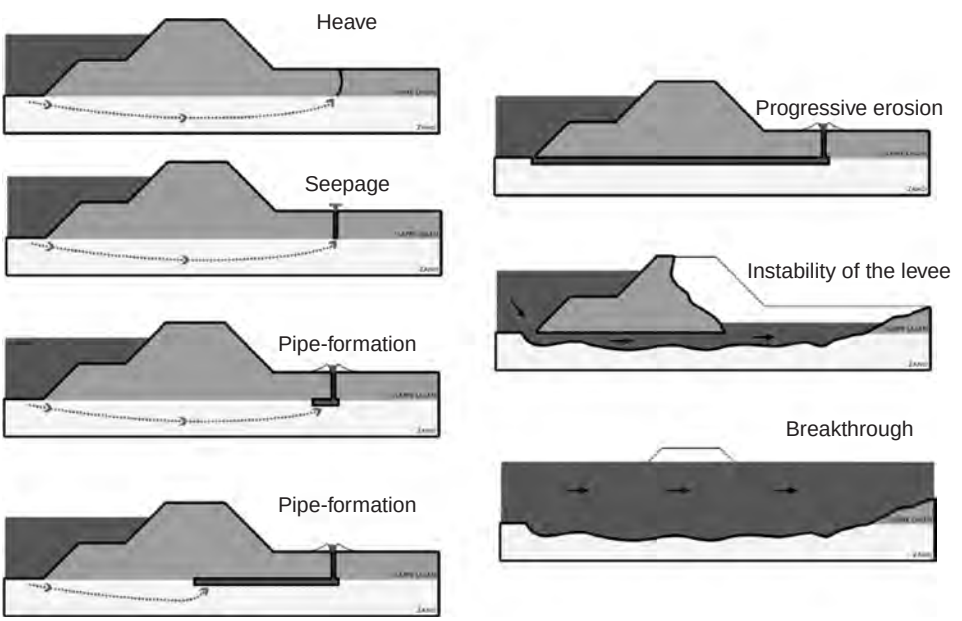


Figure 8.34 Backward erosion piping model (Sellmeijer *et al.*, 2011).



Figure 8.35 Sand boil on the land side of a levee.

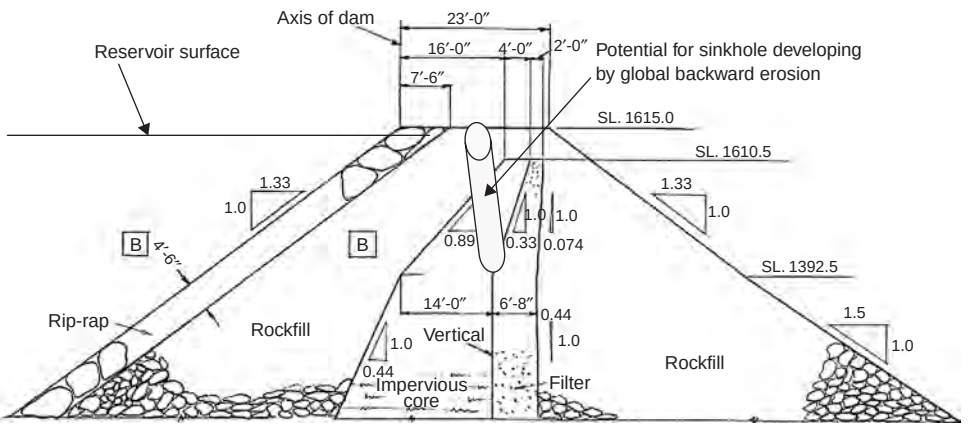


Figure 8.36 Potential for global backward erosion leading to formation of a sinkhole.

Given the high gradients required to initiate backward erosion in clay soils (see Section 8.4.7) the authors believe that only non-plastic soils are likely to be subject to backward erosion piping.

Global backward erosion, leading to development of a near vertical pipe in the core of an embankment as shown in Figure 8.36. Figure 8.37 shows another situation where this may occur.

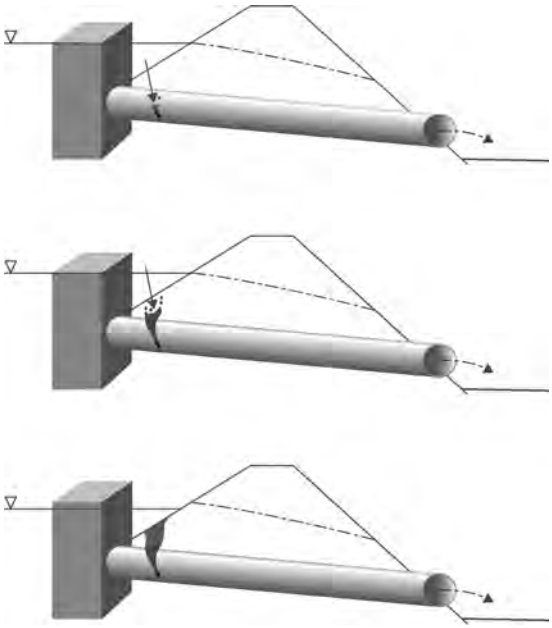


Figure 8.37 Global backward erosion into an open joint in an un-pressurized conduit through an embankment dam FEMA (2006).

Particles are detached at the downstream surface of the core which is inadequately protected by the filter or transition zone. The progression of the erosion process is assisted by gravity, and there is no need for a cohesive soil layer to form the roof for the pipe.

For practical purposes, Fell *et al.* (2008) have concluded that based on the available data, the results of Wan and Fell (2004c, 2007, 2008) tests on internal instability and their experience and judgment, soils with Plasticity Index >7 should be considered not subject to global backward erosion at the gradients experienced in dams and their foundations. This is discussed more in Section 8.5.2.

8.4.2 Experimental modelling of backward erosion piping

Sellmeijer and co-workers from Delft Hydraulics and Delft Geotechnics Laboratories in The Netherlands carried out more than 70 backward erosion piping tests. The first ones were in flumes and are reported in de Witt *et al.* (1981), Silvis (1991), Weijers and Sellmeijer (1993) and Technical Advisory Committee (1999). The tests were mostly on fine to medium sands, with a few tests on medium to coarse sands. The sands were uniform with uniformity coefficients Cu = 1.58 to 3.53. Early tests were on small scale models (base length 0.8 m), but later tests were on very large models as shown in Figure 8.38.

These experiments showed that backward erosion initiates in the slot through the strata overlying the eroding soil representing a crack or drainage ditch excavated

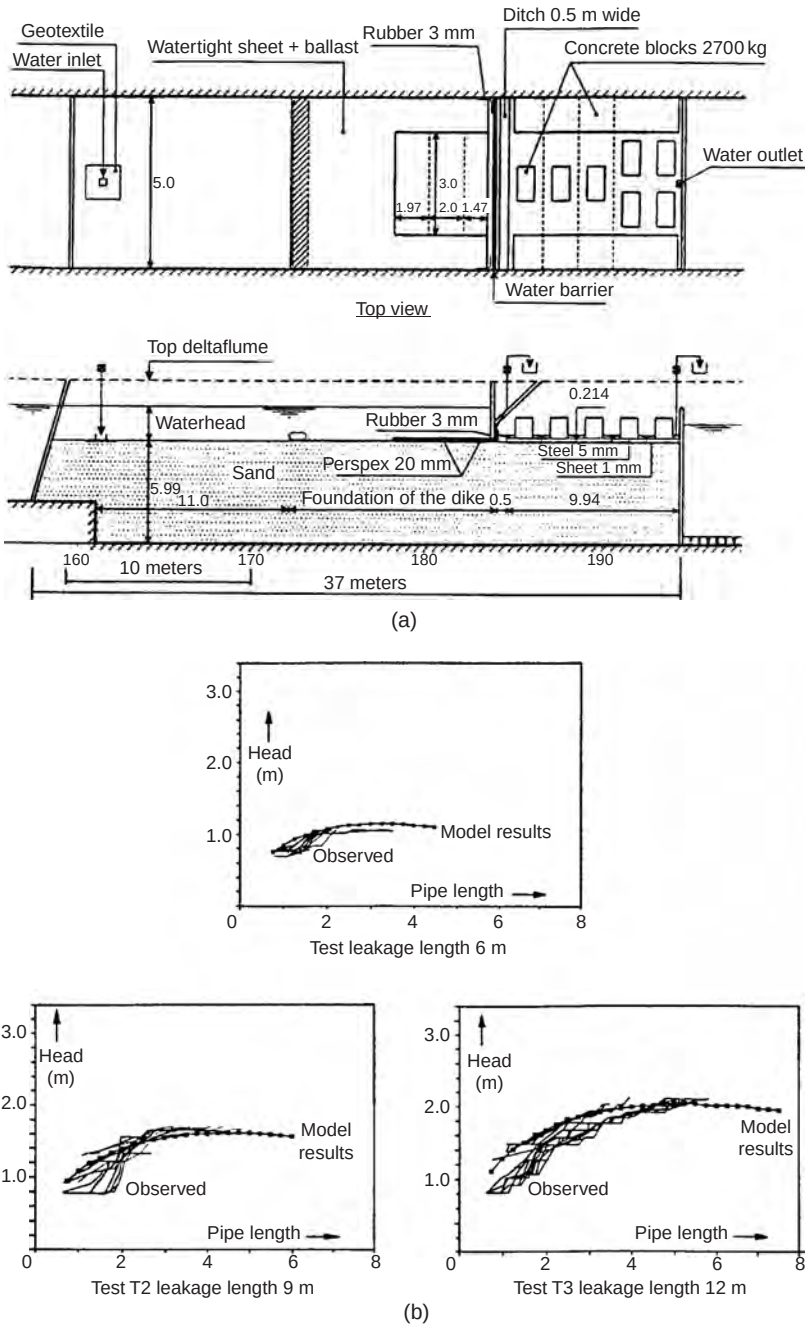


Figure 8.38 Delft Hydraulics laboratory piping flume test model (a) test equipment, (b) typical test results showing the development of the length of the tunnel as the head increases (Weijers and Sellmeijer, 1993).

through the strata, and progresses in multiple small "channels" rather than a single "pipe". Figure 8.39 shows an example of the development of the channels. The channels are quite small. The height of the channels is typically 4 to 10 (d_{15}); that is often less than 2 mm. For any head less than a critical head, the development of the channels stops. If the head is increased, erosion begins again. Figure 8.38 shows some examples. The critical head occurs when the length of the channel (l) is about 0.3 to 0.5 of the flow path length L. For heads less than this the progression of the pipe reaches a stable condition. For heads greater than the critical head the piping channel extends upstream and breaks through to the reservoir. The erosion then progresses rapidly as erosion in an open pipe. For these experiments the rate of progression of the pipe was relatively uniform until the length approaches about 40% of the total seepage path. It then accelerates. The piping progressed about 6 metres in an hour in the largest of the experiments.

In 2009 and 2010 further experiments were carried out at Deltares. These are reported in Van Beek *et al.* (2010 a, b), Sellmeijer *et al.* (2011) and Van Beek *et al.* (2012a, 2013). These were conducted at three scales as shown in Figure 8.40. The largest of these was virtually full scale. From these experiments Van Beek *et al.* (2012a, 2013) refined the backward erosion piping process as follows:

1. *Phase 1 Seepage occurs in the permeable strata*. In the experiments there were no confining strata.
2. *Phase 2 Backward erosion*. At the beginning of the process there is rearrangement of grains, individual grain movements, and formation of small channels. The process reaches equilibrium for the hydraulic head applied. Very small amounts of sand are transported, in the order of cubic centimetres in this phase of the process. With an increase in the hydraulic head to the critical head sand is transported continuously. A variety of erosion patterns is observed in the small and medium scale experiments. In the large-scale experiment sand boils are formed, and in the small and medium scale experiments craters are formed. These characterise the reaching of the critical head. The flow barely increases in this phase of the process. At a head greater than the critical head the erosion does not cease and the erosion rate is in the order of cubic decimetres per hour. The rate of erosion increases with increasing head.
3. *Phase 3 Widening of the channel*. As soon as the pipe reaches the upstream side a pressure surge occurs in the pipe. In the small-scale experiments this in turn causes a large amount of sand to be eroded rapidly. In the medium and large-scale experiments blockages caused by local collapse of the roof of the pipe take place and the widening process takes longer. The widening pipe develops from the upstream to downstream. The flow and sand transport to the exit point do not increase significantly. The widening process took up to a few days in the large-scale experiments. When the widening pipe has almost reached the downstream side the sand transport and flow increases suddenly. The situation can change from sand boils to this condition without warning.
4. *Phase 4 Failure and breakthrough*. Failure occurs soon after the widening phase is complete, but can be delayed due to collapse of the levee causing the first pipes to close.

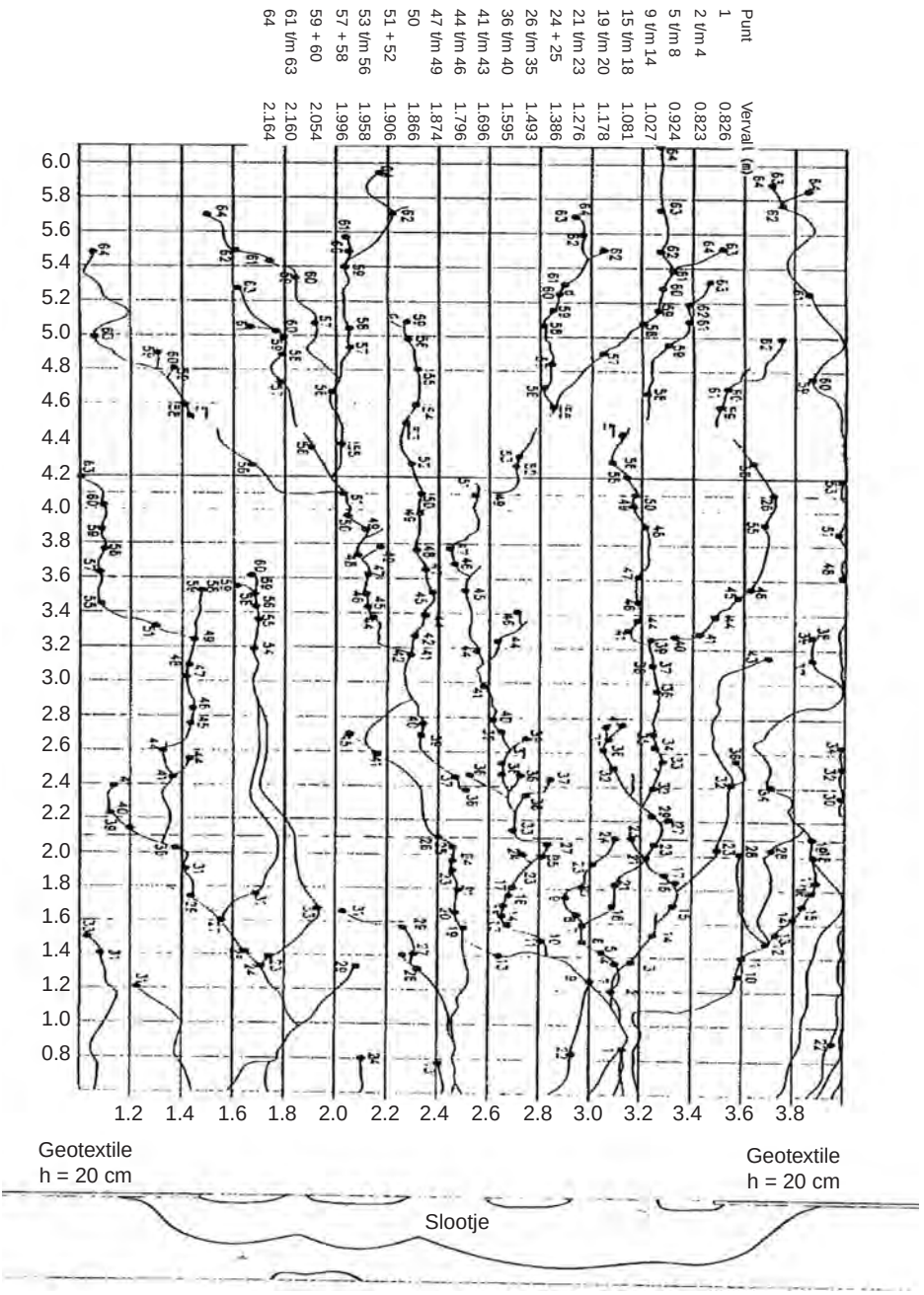
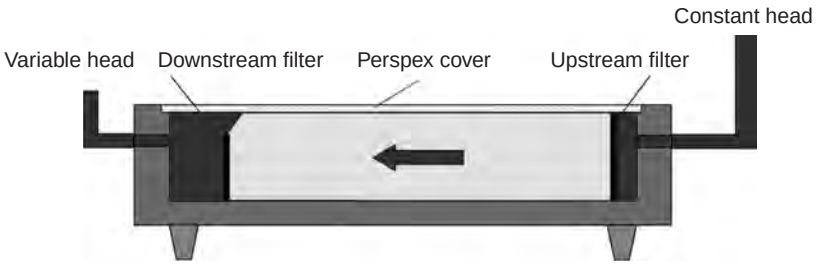


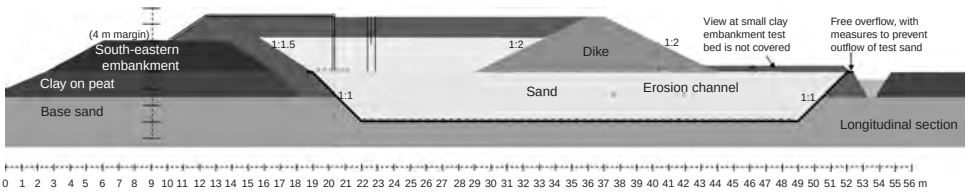
Figure 8.39 Plan view and section of an example of the development of piping channels in the Delft Experiments. The numbers are the time in minutes from when erosion initiated (Silvis, 1991).



Small scale experiments, length 0.34 metres.



Medium scale experiments, length 1.45 metres.


(4 m margin)
South-eastern embankment
Clay on peat
Base sand
1:1.5
1:1
1:2
Dike
1:2
Sand
Erosion channel
1:1
View at small clay embankment test bed is not covered
Free overflow, with measures to prevent outflow of test sand
Longitudinal section
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 m

Large scale experiments, length 15 metres.

Figure 8.40 Deltares backward erosion piping experiments (Van Beek *et al.*, 2010b, 2012).

Another extensive set of experiments were carried out at University of Florida. These are reported in Townsend *et al.* (1988) and Schmertmann (2000).

8.4.3 Methods for predicting whether backward erosion piping will initiate and progress

8.4.3.1 *Empirical rules for estimating a factor of safety*

Empirical rules for estimating a factor of safety against piping were developed by Bligh (1910) and Lane (1935). Some practitioners still use the Lane (1935) weighted line of creep method.

The method is described and discussed in Terzaghi and Peck (1948). It involves calculating the length of the path water would follow beneath a concrete structure including cut-off walls. The length is weighted so that vertical paths are weighted 3 times that of horizontal paths. The weighted creep ratio is then expressed as:

$$C_w = \frac{0.33B + \sum t}{h_{cr}} \tag{8.4}$$

where C_w = weighted creep ratio
B = horizontal seepage path length
t = vertical seepage path lengths
h_{cr} = critical head difference between the reservoir and tailwater

The critical weighted creep ratio is related to the classification and particle size of the foundation soil. These were determined from case data.

As Terzaghi and Peck (1948) discuss the problem with the method is that it is entirely empirical and there is an unknown factor of safety and that the method may give designs which may be conservative, reasonable or may lead to failure.

In view of this the authors do not recommend use of the method.

8.4.3.2 *Terzaghi and Peck (1948)*

Terzaghi and Peck (1948) show that backward erosion piping will initiate when a heave or zero effective stress condition occurs in cohesionless soils at the downstream toe of a dike, levee or dam. They do not consider the conditions under which the backward erosion will progress to form a pipe except to recognize that it is necessary for a roof to be formed in an overlying stratum or by the levee.

The Terzaghi and Peck (1948) philosophy has been the dominating influence in design of levees (dikes) and dams, particularly in the USA (e.g. USACE, 2000, 2005; Wolff, 2002). Design methods have concentrated on avoiding the heave or blow-out condition. It has however commonly been assumed, at least implicitly, that if backward erosion initiates to form a sand boil it will progress to form a pipe, at least under repeated loading from successive floods.

The Delft and University of Florida experiments show that this is not necessarily the case and that the progression of the pipe may stabilize. However the latest experiments by Deltares indicate that once sand boils form the critical gradient at which progression continue has probably been exceeded and backward erosion will progress unless the gradient is reduced by the river level dropping or the tail water level being raised, e.g. by building sand bag levees around the sand boil.

8.4.3.3 *Sellmeijer and co-workers at Deltares method*

Sellmeijer (1988), Sellmeijer and Koenders (1991), Koenders and Sellmeijer (1992) developed a mathematical model for backward erosion piping based on the experiments carried out at Delft Laboratories in the 1980's.

This model was refined using the results of the Deltares testing and is presented in Van Beek *et al.* (2012a, 2013) and Sellmeijer *et al.* (2011).

The critical gradient is determined as a product of three contributions: resistance factor, scale factor and geometrical shape factor. The improvement is realized by the outcome of 38 small-scale tests, applying the multivariate method. The refined equations for the critical gradient at which backward erosion will progress are:

$$\frac{H}{L} = \frac{1}{c} = F_R F_S F_G \tag{8.5}$$

$$F_R = \eta \frac{\gamma'_p}{\gamma_w} \tan\vartheta \left(\frac{RD}{RD_m}\right)^{0.35} \left(\frac{U}{U_m}\right)^{0.13} \left(\frac{KAS}{KAS_m}\right)^{-0.02} \tag{8.6}$$

$$F_S = \frac{d_{70}}{\sqrt[3]{\kappa L}} \left(\frac{d_{70\,m}}{d_{70}}\right)^{0.6} \tag{8.7}$$

$$F_G = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8}-1}+0.04} \tag{8.8}$$

where

H [m]:	hydraulic head across structure
L [m]:	seepage length (= base length of the embankment)
D [m]:	thickness of sand layer under the embankment
c [−]:	erosion coefficient
F_R [−]:	resistance factor
F_S [−]:	scale factor
F_G [−]:	geometrical shape factor
RD [%]:	relative density
U [−]:	uniformity coefficient $Cu = d_{60}/d_{10}$
KAS [%]:	roundness
d_{70} [m]:	soil particle diameter for which 70% by weight of the soil is finer
γ'_p [kN/m^3]:	submerged unit weight of soil particles 9.8(G-1)
G [t/m^3]:	soil particle density
γ_w [kN/m^3]:	unit weight of water
η [−]:	Whites drag coefficient
ϑ [deg]:	bedding angle (angle of repose) of sand
K [m^2]:	intrinsic permeability

where: $K = \frac{\upsilon}{g} k$

υ [m^2/sec]:	kinematic viscosity
g [m/sec^2]:	gravity
k [m/sec]:	hydraulic permeability

H, D and L are defined in Figure 8.41. In these equations the variables are normalized by the mean values in the data set. For the data set used by those authors the mean values are as detailed in Table 8.14.

Sellmeijer *et al.* (2011) indicate that their data set assumes a bedding angle of 37 degrees and a Whites coefficient of 0.25. For water at 20° Celsius, $K = 1.02 \times 10^{-7}$(k)

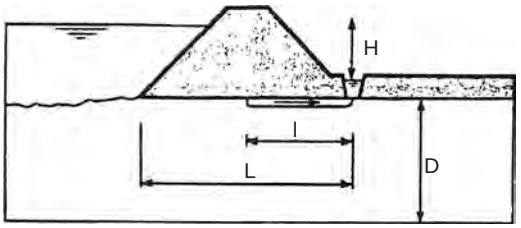


Figure 8.41 Geometry of Delft backward erosion piping model (Koenders and Sellmeijer, 1992).

Table 8.14 Limits of the Sellmeijer *et al.* (2012) method.

Parameter	*Minimum*	*Maximum*	*Mean*
RD	34%	100%	72.5%
U	1.3	2.6	1.81
KAS	35%	70%	49.8%
d_{70}	150 μm or 1.5E–04 m	430 μm or 4.3E–04 m	207 μm or 2.07E–04 m

where K is m^2, k is in m/sec. They indicate that they ignore the roundness and Uniformity Coefficient terms as they do not contribute significantly.

They indicate that the refinements have been determined from the small-scale tests and it is not altogether clear if there may be a scale effect, so that the outcome for large structures may not be properly modeled. They also indicate that the equations should only apply within the limits of the parameters during testing. These limits are given in Table 8.14.

They found that the equations predicted the large scale experiment behavior quite well for the fine grained soil, but that it was not so accurate for the soil with a $d_{70} = 260\,\mu m$.

There are very significant scale effects. The smaller the structure, the higher the critical gradient. In the tests the value of the scale factor F_S ranged from 0.134 for the large-scale IJkdijk tests, to 0.192 for the medium-scale tests and 0.421 for the small-scale tests.

It should be noted that Sellmeijer *et al.* (2012) emphasise that the above equations do not include any margin of safety and that for design rules they are likely to apply factors of safety.

Van Beek *et al.* (2012b) consider the case where the soil subject to backward erosion piping is underlain by coarser sand strata as shown in Figure 8.42.

They indicate that Equations 8.5 to 8.8 apply provided that:

The permeability k, is replaced by $k_{h,avg}$ where

$$k_{h,avg} = \sum_{m=1}^{n} \frac{k_{h,m} D_m}{D_{tot}} \tag{8.9}$$

$k_{h,m}$ = Horizontal permeability of layer m [m/sec]
D_m = Thickness of layer m [m]
D_{tot} = Total thickness of m layers [m]

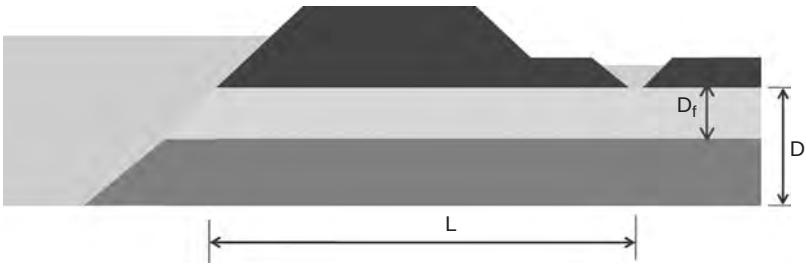


Figure 8.42 Geometry of layered foundation model (Van Beek *et al.*, 2012).

Provided $D/L < 0.3$ and $k_c/k_f < 10$

where

k_c = Horizontal permeability of the coarser layer
k_f = Horizontal permeability of the finer layer.

8.4.3.4 *Schmertmann method*

Schmertmann (2000) developed a method which is based on backward erosion piping tests in flumes at University of Florida (U of F). These were carried out in a number of sets, including those by Townsend *et al.* (1988) who carried out 15 tests. Those tests were carried out on a range of soils from fine to medium sands, up to coarse sand/fine gravel mixes.

The tests were different to the Delft tests in that a starter pipe was formed at the downstream end. The head differential was increased progressively to keep the tunnel progressing upstream until the critical head was reached causing the tunnel to form a pipe to the upstream source.

Most of the tests (32 out of 39) which Schmertmann (2000) relied upon, were carried out on sands with a uniformity coefficient less than 3.2. The other 7 tests were carried out on gap-graded soils and a well-graded soil with uniformity coefficients around 6.

Schmertmann (2000) found that the critical average gradient $\bar{i}_{pmt}$ was related to the uniformity coefficient Cu (d_{60}/d_{10}) of the soils tested. He also plotted the Delft tests and found a similar correlation. The U of F test data is shown in Figure 8.43. However there were few tests on soils with a $Cu > 3$.

It is notable that two of the data sets with Cu around 6 were gap graded. The test descriptions in Townsend *et al.* (1988) indicate fine sand did erode in these tests so the interpretation of the tests is somewhat complicated. These tests and two tests on well-graded sand with a d_{60} of 1.6 mm withstood a gradient of greater than 1 without developing a pipe.

The test geometries used at University of Florida and Delft were not the same, so Schmertmann (2000) applied correction factors for geometry and was then able to plot all the results together to give Figure 8.43 with his recommended design line.

The method involves quite large corrections for scale effects. Given it is based on little data in the larger uniformity coefficient range, and some of these may be affected

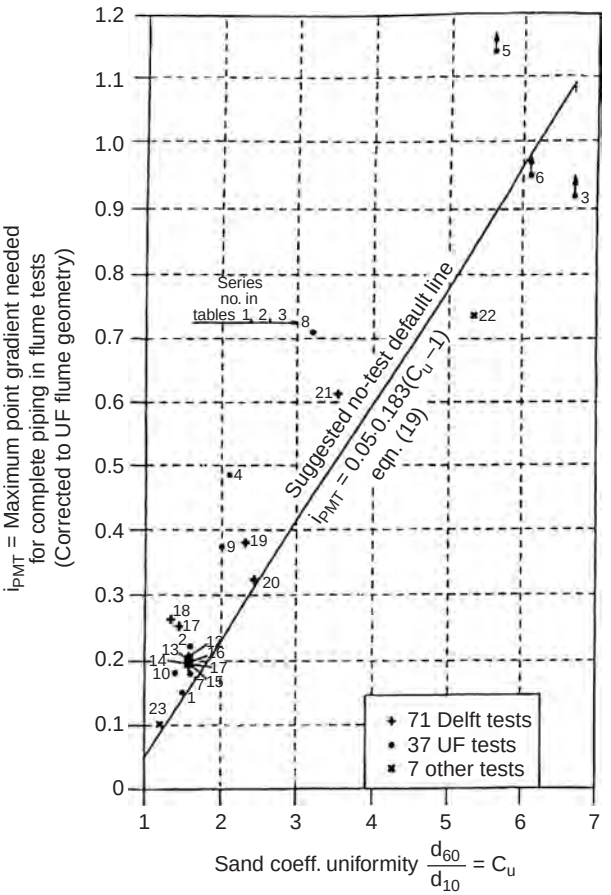


Figure 8.43 Maximum point gradient, i_{pmt}, needed for complete piping (initiation and progression for an unfiltered exit) versus uniformity coefficient of soil (Schmertmann, 2000).

by internal instability, caution should be exercised in using the method for soils with Cu > 3 and for important decisions laboratory tests carefully designed to model the accrual conditions should be carried out.

Further research is under way at UNSW on broadly graded soils. This is expected to result in improved methods of prediction of critical gradients for such soils.

8.4.4 Some field observations

The USACE have responsibility for managing the levee systems along major USA Rivers including the Mississippi. They have carried out extensive studies of backward erosion piping over many years. Some observations from these include:

In any one flood the levee system in one USACE District, e.g. St Louis, may experience many hundreds of sand boils in the foundations. However there are few



(a)



(b)



(c)

Figure 8.44 Measures to control backward erosion piping (a) Ringing single sand boil with sand bags; (b) Ringing several sand boils; (c) Pressure relief well (Photographs courtesy of George Sills and Danny McCook).

cases of breaching of the levees from piping. This is influenced by the "flood fighting" efforts of the Corps and the Levee District personnel. As shown in Figure 8.44 they build sand bags and in some cases sub-levees around the sand boils to stop the flow of sand (but not the flow of water). For longer term control pressure relief wells are constructed.

Sills and Vroman (2007), Wolff (2002) and Glynn and Kusmaul (2004) report that there are cases of levees which have sand boil activity occurring at successively lower average gradients (lower river stages). This phenomenon does not appear to have been investigated by the laboratory flume tests described above.

Glynn and Kusmaul (2004) show that greater sand boil activity occurred in the 1995 flood than the 1993 flood, even though the river stage was lower in 1995. It is not clear why this is so, but it may relate to the duration of the flood and the time it takes to set up a seepage flow net.

USACE (1956) and Wolff (2002) show that local geology has an important influence on the occurrence of sand boils. Sand boils are more likely to occur where swales from point bar deposits cross the levee at an angle and concentrate seepage at the toe.

Barrett and Bliss (2008) describe a piping incident in the foundation of AV Watkins dam. This incident was attributed to backward erosion piping through fine sand and silty sands deposits within the foundation. Piping had initiated where the silty sand and sand layers day-lighted in the sides of a drainage ditch located downstream of the embankment. The roof of the pipe was formed by a thin hardpan layer. Backward erosion was able to progress below the embankment under a low average seepage gradient of about 0.05 (1 in 20). Backward erosion piping progressed and reached the reservoir, however emergency remedial measures prevented the dam breaching.

8.4.5 Suggested approach to design for and assessing backward erosion piping

The authors suggest that in view of the limitations of the available methods the following approach be adopted:

- So far as practicable, design so that initiation of backward erosion is prevented by preventing heave by increasing seepage path lengths or using pressure relief wells or cut-offs. That is, follow the USACE approach that any sand boil is a bad sand boil.
- If there is a free exit for erosion, so "heave" is not required for initiation, provide a filtered exit sufficiently thick to give a factor of safety against heave or install pressure relief wells to reduce seepage gradients.
- If the soils satisfy the requirements of the Sellmeijer *et al.* method use it, but apply factors of safety to cover the uncertainty in scaling and input parameters.
- Do not use the Schmertmann method.

8.4.6 Guidance on whether the overlying soil will form a roof to the pipe

See Section 8.8.3. Table 8.16 may be used to give guidance on whether the overlying strata will form a roof for the backward erosion pipe.

8.4.7 Methods for prediction of initiation and progression of global backward erosion

The available evidence is that unless the core is non-plastic internally unstable soil subject to suffusion, then global backward erosion will only occur at relatively high gradients. This evidence includes:

(a) Laboratory tests by Sun (1989), Marot *et al.* (2007) and Bendahmane *et al.* (2008) confirmed that global backward erosion could occur in more cohesive soils, but initiated at very high gradients which were not likely to occur in dams or their foundations.
(b) Bendahmane *et al.* (2008) carried out tests which showed that soils consisting of 10% kaolin/90% fine sand initiated global backward erosion at gradients between 90 and 140 depending on confining pressure. For kaolin contents 20% and 30% no backward erosion occurred at gradients as high as 100. They observed what they called suffusion with gradients as low as 5, but noted that this value should not be taken as generally applicable.
(c) Moffat and Fannin (2011) and Moffat *et al.* (2011) indicate that there is an initial movement of finer particles at gradients of about 4 for the cohesionless soils they tested and more extensive movement of finer and some coarser fraction soils at higher gradients, from about 10 to 30 in the soils tested. These movements of particles are accompanied by a reduction in volume as particles are eroded from the soil through inadequate filters.
(d) Marot *et al.* (2007) and Bendahmane *et al.* (2008) tested clay soils and Sail *et al.* (2011) a non plastic gap graded soils with 40% finer fraction. They observed some particle movement at a gradient of about 5 and major movement at much higher gradients.

Hunter *et al.* (2012) carried out tests on glacial till soils from an Australian dam. Two samples were tested; "fine test grading" and "coarse test grading" with gradations as shown in Figure 8.45. The test set up was for vertical downward flow to model the condition in Figure 8.36. The sample was supported on a coarse sieve sufficient to allow excessive erosion as defined by Foster and Fell (2001).

The fine sample did not erode under gradients as high as 9. Figures 8.46 and 8.47 show the cumulative sample loss with time, and seepage rate with time.

Global backward erosion occurred in the test at a gradient of 9 after erosion had progressed for about 40 days at a gradient of 5 and may have reached the failure condition without the increased gradient if the test had gone longer. The gradation of the eroded material indicated that the erosion was backward erosion as particles ranging in size up to the maximum opening of the supporting sieve mesh were present.

However the sample plots as marginally internally unstable in the Wan and Fell (2008) method shown in Figure 8.55 and this may be important.

This testing raises questions as to time effects for this type of global backward erosion as it seems the process may be very slow. More testing is planned at UNSW to explore this phenomenon.

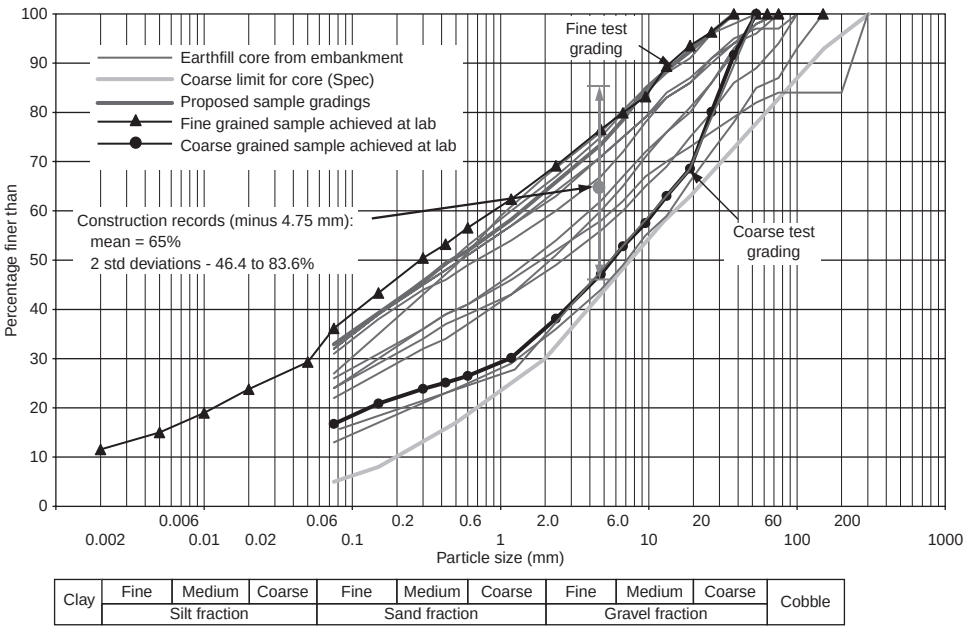


Figure 8.45 Sample gradings for global backward erosion testing (Hunter *et al.*, 2012).

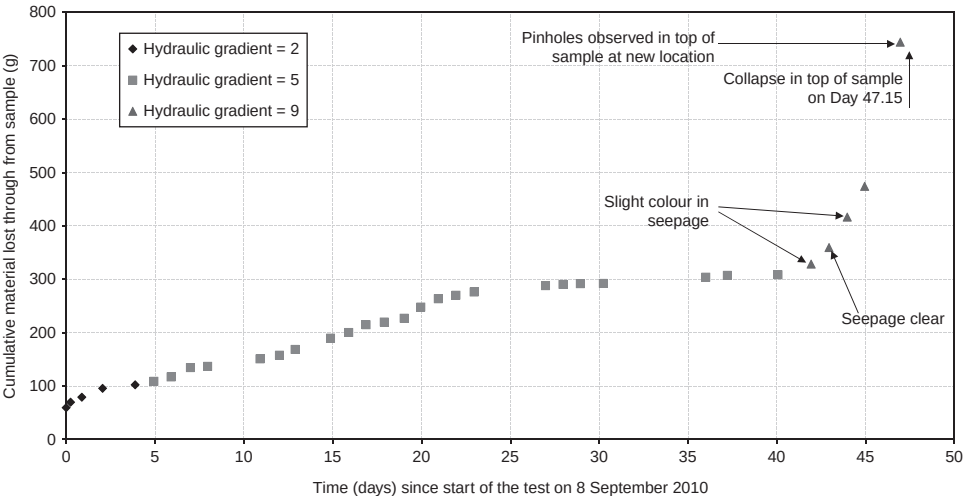

Hydraulic gradient = 2
Hydraulic gradient = 5
Hydraulic gradient = 9
Pinholes observed in top of sample at new location
Collapse in top of sample on Day 47.15
Slight colour in seepage
Seepage clear
Cumulative material lost through from sample (g)
Time (days) since start of the test on 8 September 2010

Figure 8.46 Cumulative sample losses with time for test on coarse grained sample (Hunter *et al.*, 2012).

There are currently no known reliable methods for predicting global backward erosion in broadly graded cohesionless soils such as glacial tills in the cores of dams and for development of vertical downward piping as shown in Figure 8.36. For these situations it is recommended that specifically designed laboratory tests should be carried out.

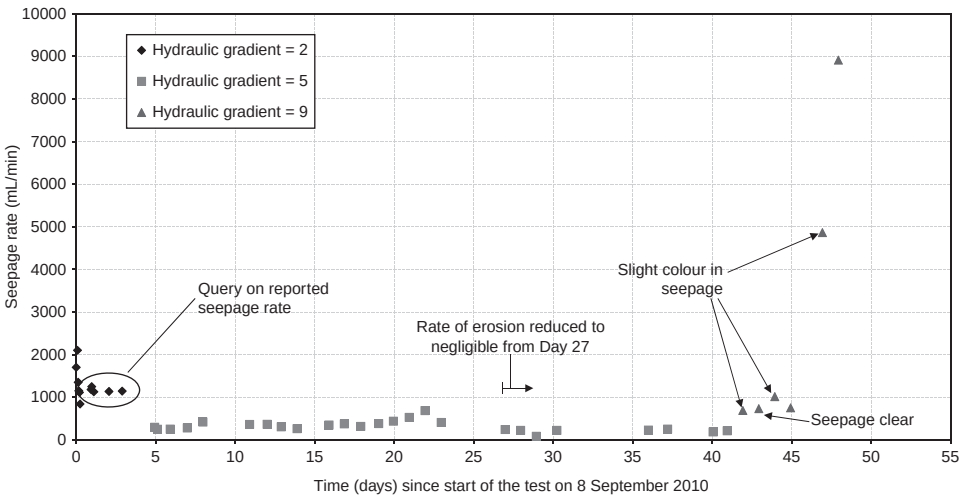

Hydraulic gradient = 2
Hydraulic gradient = 5
Hydraulic gradient = 9
Seepage rate (mL/min)
Time (days) since start of the test on 8 September 2010
Query on reported seepage rate
Rate of erosion reduced to negligible from Day 27
Slight colour in seepage
Seepage clear

Figure 8.47 Seepage versus time for test on coarse grained sample (Hunter *et al.*, 2012).

8.5 SUFFUSION OF INTERNALLY UNSTABLE SOILS

8.5.1 General description of suffusion

Suffusion occurs when water flows through internally unstable widely graded or gap graded cohesionless soils such as alluvium of a large river, colluvium in the bed of rivers in mountainous areas, embankment cores constructed of glacial origin soils and in filters which have very broad or gap gradings or an excessive fines content.

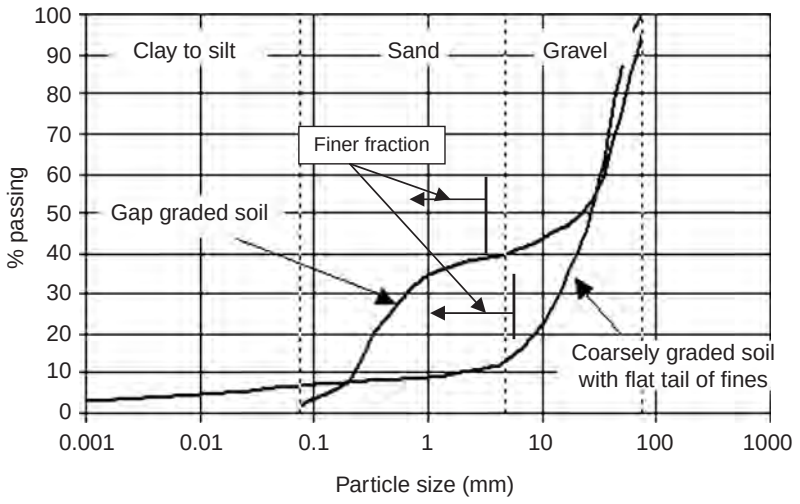
Coarse graded and gap graded soils, such as those shown schematically in Figure 8.48 are potentially internally unstable and susceptible to suffusion.

The phenomenon does not require a crack within the soil in which erosion may occur as is required in concentrated leak erosion, or a free surface from which particles detach and a roof of cohesive soil as is required for backward erosion.

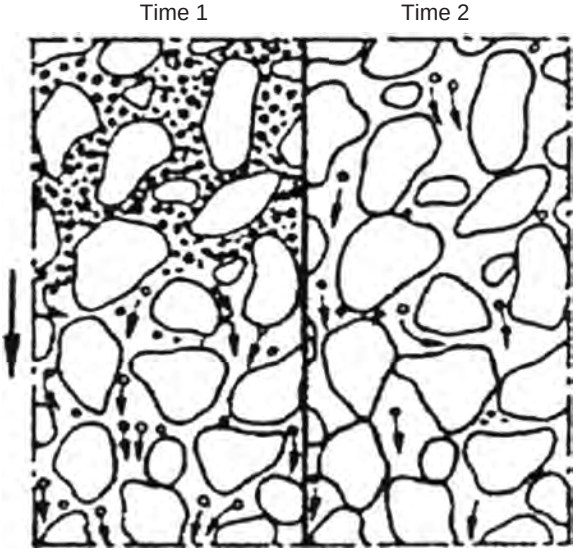
The small particles of soil are transported throughout the soil by the seepage flow through the pores of the coarser particles. The coarser particles are not transported and the effective stresses are largely transferred through the matrix of the coarser particles.

For suffusion to occur, the following three criteria, geometric criterion, stress criterion and hydraulic criterion, have to be satisfied (Wan and Fell, 2008):

- Criterion 1: The size of the finer soil particles must be smaller than the size of the constrictions between the coarser particles, which form the basic skeleton of the soil.
- Criterion 2: The amount of finer soil particles must be less than enough to fill the voids of the basic skeleton formed by the coarser particles. That is the voids must be "under-filled". If there are more than enough finer soil particles for void filling, the coarser particles will be "floating" in the matrix of fine soil particles, instead



(a)

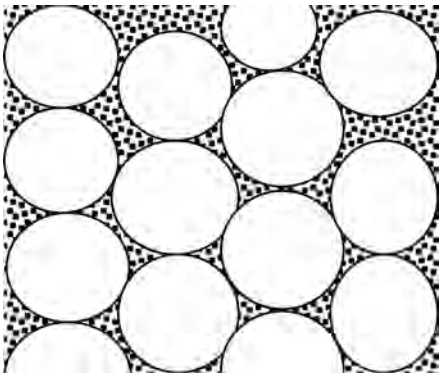


(b)

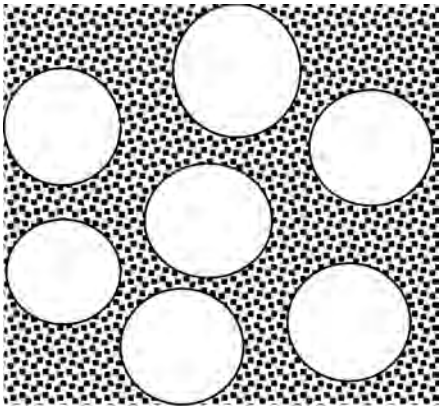
Figure 8.48 (a) Soil gradation types which are potentially internally unstable and susceptible to suffusion. (b) Schematic of suffusion process (Ziems, 1969).

of forming the basic soil skeleton. This is termed "over-filled" voids. Figure 8.49 shows this schematically.

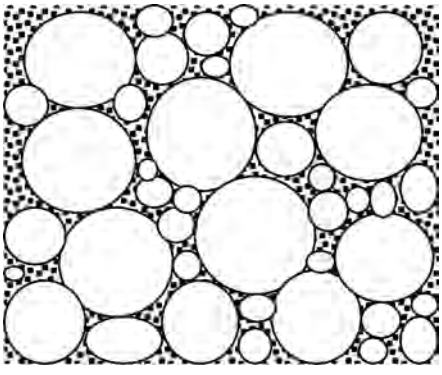
- Criterion 3: The velocity of flow through the soil matrix must impose a high enough stress to overcome the stresses imposed on the particles by the surrounding soil and to move the finer soil particles through the constrictions between the larger soil particles.



(a) Internally unstable soil with under-filled voids



(b) Internally stable gap graded soil with overfilled voids



(c) Internally stable well graded soil

Figure 8.49 Schematic sketches of internally unstable and stable soils.

Suffusion results in an increase in permeability, greater seepage velocities, and potentially higher hydraulic gradients, possibly accelerating the rate of suffusion. Stable situations may be reached with some of the finer fraction remaining in equilibrium with the seepage stresses. Suffusion may re-commence during cycling periods of water loads or during higher reservoir or river water level.

Suffusion occurring within an embankment core or the foundation of a dam may also lead to some settlement of the embankment.

A filter constructed of internally unstable materials will have a potential for erosion of the finer particles in the filter, rendering the filter coarser and less effective in protecting the core materials from erosion.

Some authors have used the terms suffusion and suffosion to describe internal erosion in global backward erosion laboratory experiments on internally stable cohesionless soils and cohesive soils under very high gradients. This includes the tests by Moffat and Fannin (2011), Moffat *et al.* (2011), Marot *et al.* (2007), Bendahmane *et al.* (2008) and Sail *et al.* (2011)

These soils have over-filled voids between the coarser particles, so the finer fraction is not free to move under the gradients normally experienced in dams.

The phenomena observed are not suffusion as defined in this book. It may be concentrated leak erosion due to hydraulic fracture or a form of global backward erosion.

8.5.2 Methods of identifying soils which are internally unstable and potentially subject to suffusion

8.5.2.1 *General requirements*

Figures 8.50 and 8.51 show particle size distributions of some soils which have been found to be internally unstable in laboratory tests. It will be noted that there are gap graded and broadly graded soils. The Kenney and co-authors soils are sandy gravels, whereas the Wan and Fell soils are silty sandy gravels. Soils 14A and 15 had 11% and 21% kaolin in them so were slightly plastic.

As pointed out by Kenney and Lau (1985, 1986) for a soil to be internally unstable and subject to suffusion the percentage of finer fraction (finer than the point of inflection of the particle size plot) must be smaller than the available void space. They suggested this lay between 20% of the total soil for well graded soils and 30% for narrow graded soils. Wan and Fell (2004c, 2007) showed that this could theoretically be as high as 40% but in their samples it was between 22% and 33% for broadly graded soils and 29% and 38% for the gap graded soils.

For a greater percentage of finer soil the coarse particles are surrounded by the finer particles. That is the voids are over-filled. Such soils are not subject to suffusion but may be subject to global backward erosion.

For practical purposes Fell *et al.* (2008) and Fell and Fry (2013) have concluded that based on the data described, available soils with Plasticity Index > 7 should be considered not subject to suffusion at the gradients usually experienced in dams and their foundations. If for some particular reason the gradient is higher than about 4 then soils with a Plasticity Index ≤ 12 should be considered for suffusion. This was considered to be a somewhat conservative approach.

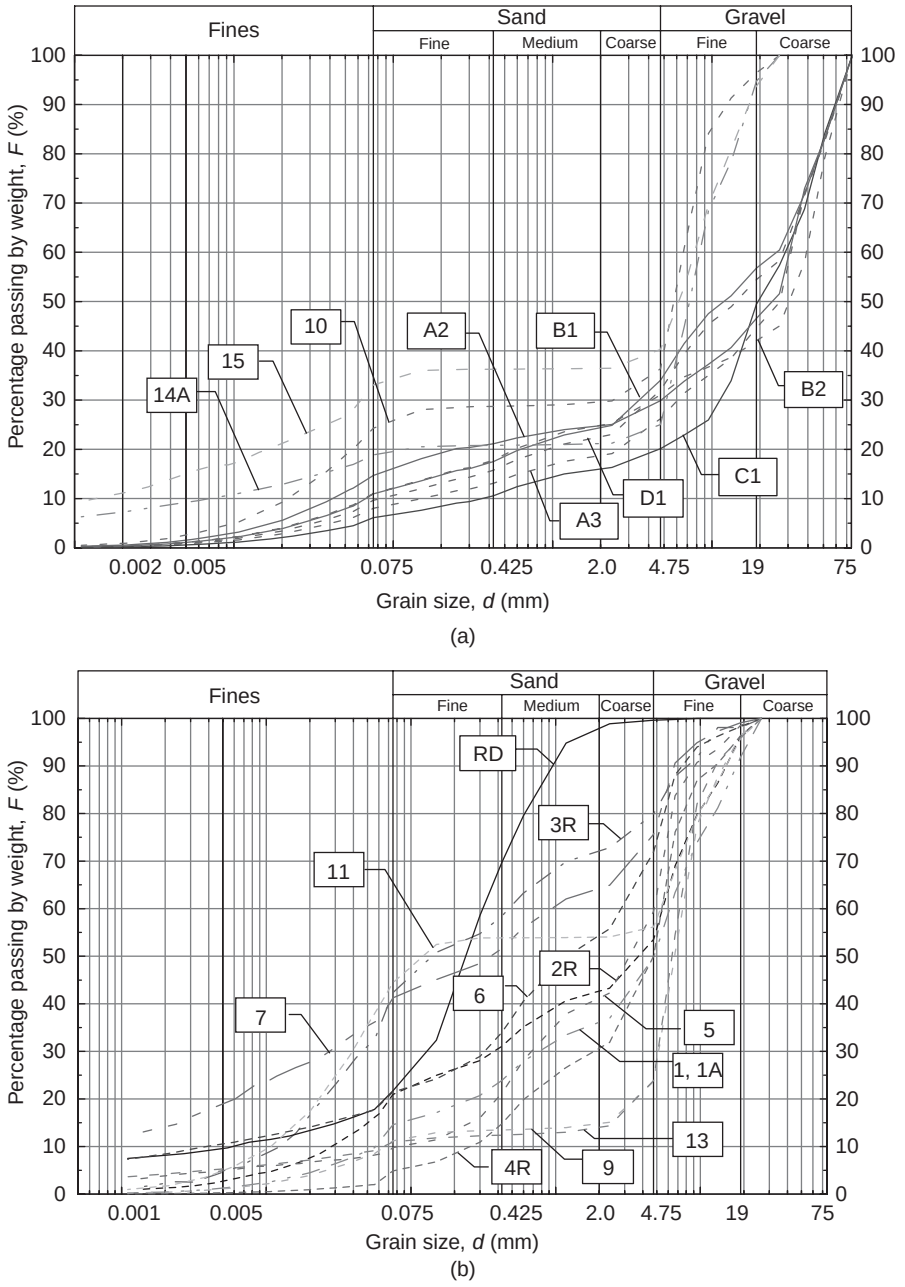


Figure 8.50 (a) Soil samples tested as being internally unstable (suffusive) by Wan and Fell (2004c, 2007). (b) Soil samples tested as being internally stable (not suffusive) by Wan and Fell (2004c, 2007).

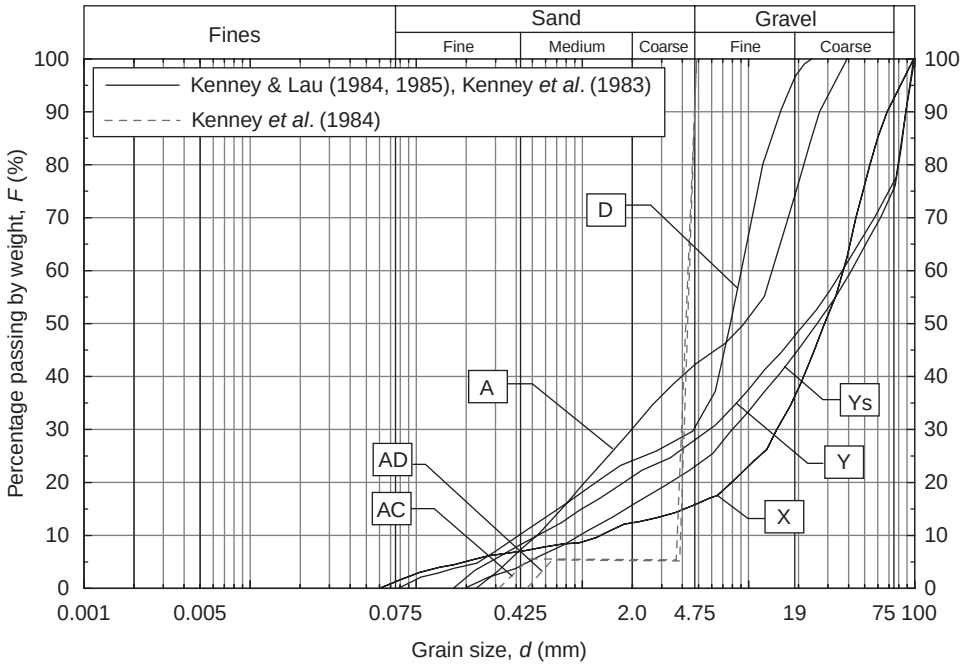


Figure 8.51 Soil samples tested as being internally unstable (suffusive) by Kenney & Lau (1984, 1985) and Kenney *et al.* (1985).

8.5.2.2 *Some methods for assessing whether a soil is internally unstable*

There are a number of methods available to determine whether a soil is internally unstable and therefore potentially subject to suffusion. The following are some of the more widely used and / or later methods. It is suggested that the method or methods which were developed for soils most closely matching the soil being assessed to be used. For important decisions it may be necessary to carry out tests on the soils under consideration given the uncertainty in the methods currently available.

Kenney and Lau method

The Kenney and Lau (1985, 1986) method plots F, the mass fraction smaller than versus grain size D, (as in a conventional particle size distribution plot) for the soil and from this a plot of F versus H, the mass fraction between D and 4D. For particles to move there must be a deficiency in the mass of particles in the range D to 4D. Figures 8.52 and 8.53 show the method.

If the soil plots to the right of the boundary of Figure 8.53 ($H < F$), it is likely to be internally unstable provided that it satisfies the rules on limiting finer fraction; that is $F \leq 0.2$ for widely graded soils and $F \leq 0.3$ for narrow graded soils.

The method was developed for filter and transition zones with less than 5% fines passing 0.075 mm.

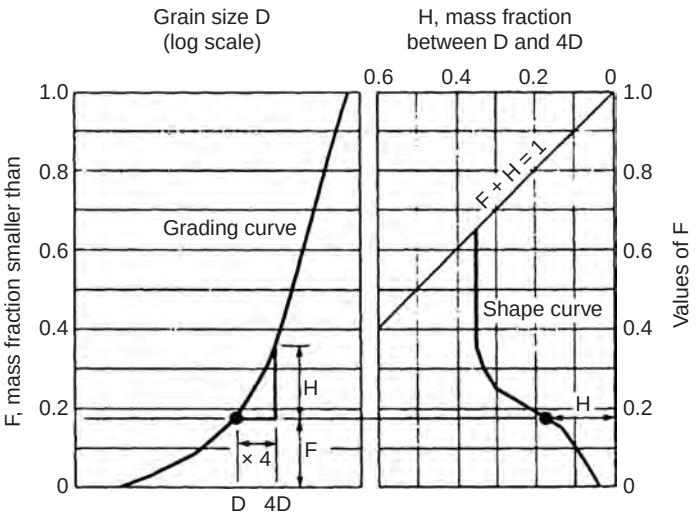


Figure 8.52 Method of characterising the shape of the gradation curve (Kenny and Lau, 1985).

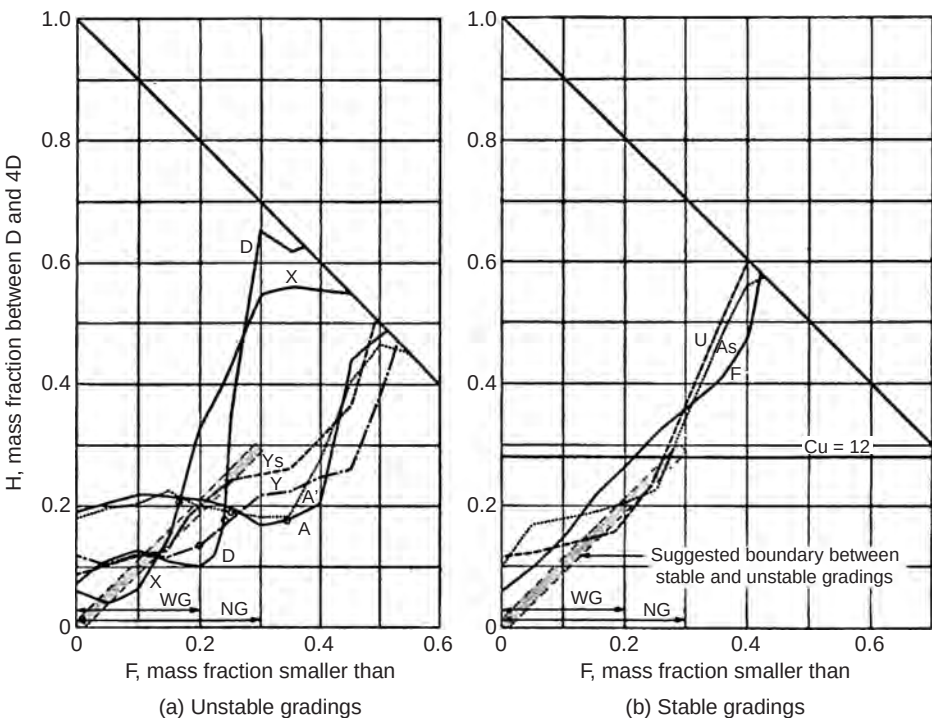


Figure 8.53 Method of assessing internal instability by Kenney and Lau (1985), with the revised criteria from Kenney and Lau (1986). Legend: WG, limit of internally unstable widely graded soils (Cu > 3); NG, limit of internally unstable soils narrowly graded (Cu < 3).

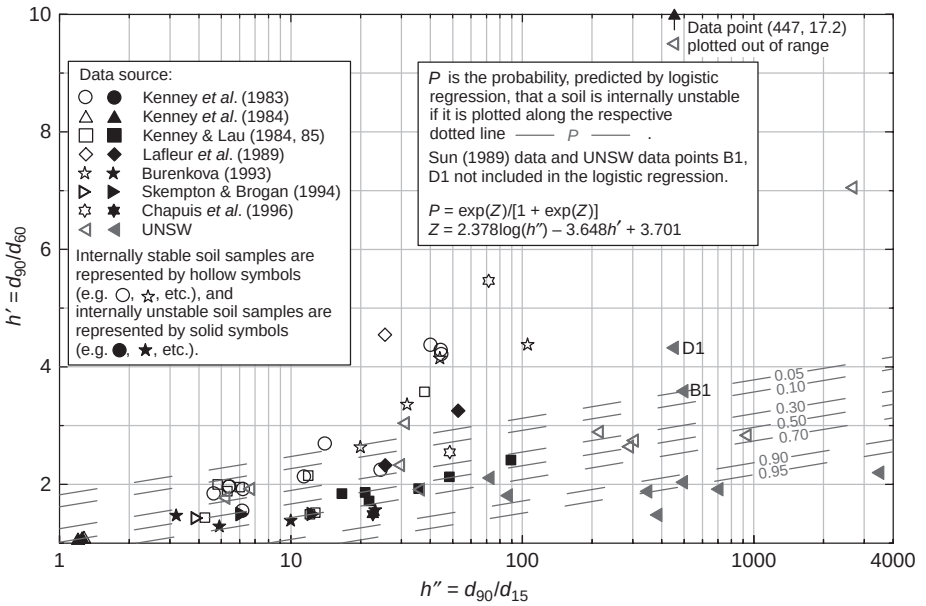


Figure 8.54 Contours of the probability of internal instability (potentially suffusive) for silt-sand-gravel soils and clay-silt-sand-gravel soils of limited clay content and plasticity (Wan and Fell, 2004c, 2007).

Wan and Fell adaptation of the Burenkova method

Wan and Fell (2004c, 2007, 2008) found that the Burenkova (1993) method gave reasonable assessments of whether a soil was internally unstable when used for the soils they had tested. However the method does not give a clear-cut boundary between internally stable and unstable soils in the data set. To model this uncertainty, logistic regression was used by Wan and Fell (2004c, 2007) to define contours of equal probability of internal instability. Figures 8.54 and 8.55 show the contours and the logistic equations.

Figure 8.54 is to be applied to silt-sand-gravel and clay-silt-sand-gravel mixtures with a plasticity index less than 13% and less than 10% clay size fraction (% passing 0.002 mm) and Figure 8.55 to sand-gravel soils with less than 10% non-plastic silt fines passing 0.075 mm.

8.5.2.3 *Some general comments*

Wan and Fell (2004c, 2007) found that for the silty, sandy gravel soils they tested, the Kenney and Lau (1985, 1986) method was too conservative, identifying the soils shown in Figure 8.50(b) as internally unstable when they tested as stable. In view of this it is apparent that the Kenney and Lau (1985, 1986) method is too conservative for silty, sand gravel soils and it is suggested it only be applied to the sand gravel soils for which it was developed. Wan and Fell (2004c, 2007) also found that the Istomina (1957), Sherard (1979) and Sun (1989) methods were conservative for these silt, sand gravel soils.

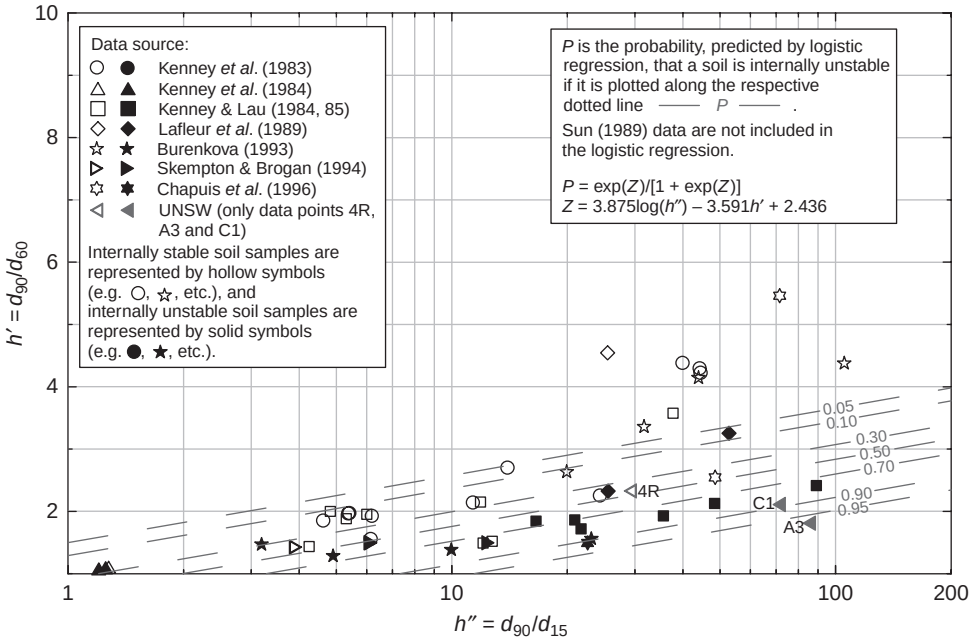


Figure 8.55 Contours of the probability of internal instability potentially suffusive for sand-gravel soils with less than 10% non-plastic fines passing 0.075 mm (Wan and Fell, 2004c, 2007).

8.5.3 Assessment of the gradation after suffusion

Wan (2006) and Wan and Fell (2004c) give details of a method to determine what fraction of the soil which will be eroded.

Fell *et al*. (2008) have suggested that in practical terms it can be assumed that 50% of the finer fraction as defined by the point of inflection of broadly graded soils and the fine limit of the gap in gap-graded soils is eroded, and the particle size distribution re-plotted as shown in Figure 8.56.

They suggest that if the gradation after suffusion is critical (e.g. for assessing the D_{15F} of an internally unstable filter or transition material), then laboratory tests should be carried out on the internally unstable soil.

Salehi Sadaghiani and Witt (2011) also found that about 50% of the finer fraction eroded in gap graded soils.

8.5.4 Assessment of the seepage gradient which will cause suffusion

Skempton and Brogan (1994) show that erosion will begin in internally unstable cohesionless soils at seepage gradients lower than the Terzaghi critical or zero effective stress gradient. Wan and Fell (2004c, 2007) found that internally unstable soils tested erode with upward gradients of 0.8 or less, with several samples tested at less than 0.3.

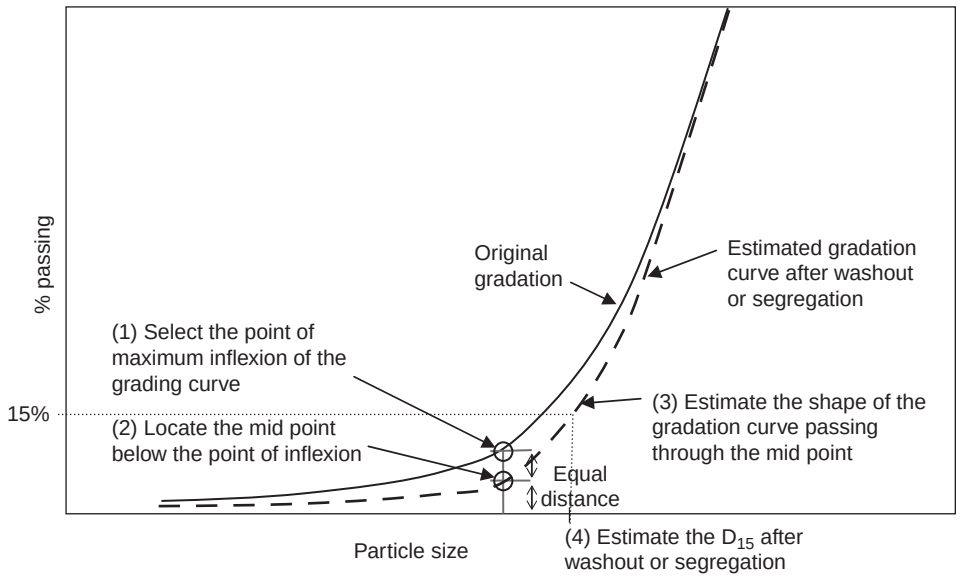

% passing
15%
Original gradation
Estimated gradation curve after washout or segregation
(1) Select the point of maximum inflexion of the grading curve
(2) Locate the mid point below the point of inflexion
(3) Estimate the shape of the gradation curve passing through the mid point
Equal distance
Particle size
(4) Estimate the D_{15} after washout or segregation

Figure 8.56 Approximate method for estimating the gradation and D_{15} of an internally unstable soil after washout of the erodible fraction from an internally unstable soil (Fell *et al.*, 2008).

They found that there is a general trend that soils with a higher porosity begin to erode at lower hydraulic gradients; loose, higher porosity soils tested began to erode at gradients less than 0.3; soils with plastic fines required higher gradients to begin to erode, and gap-graded soils tended to begin to erode at lower gradients than non gap-graded soils with the same fines content.

Li and Fannin (2012) proposed a hydro-mechanical criterion deduced from tests on unstable materials. Starting from the fact that seepage failure occurs in stable soils under the Terzaghi critical hydraulic gradient, i_{CT}, they deduced that the critical gradient is dependent on the effective stress.

Their data included tests on internally stable soils and for these soils their conclusions seem reasonable. For internally unstable soils the effective stress should not be a significant factor because the coarse particles should carry the load.

The Li and Fannin (2012) approach lacks sufficient test data to be relied upon.

8.5.5 Some general comments

8.5.5.1 *Need for project specific laboratory tests*

As there are no generalized methods for accurately predicting whether a soil will be internally unstable, and the critical seepage gradient, laboratory tests may be needed to be carried out which carefully simulate the field conditions. It may be necessary to construct specific test equipment because of the large particles involved.

8.5.5.2 *Do not use "average" soil gradations*

In view of the fact that quite minor differences in gradation may render a soil internally unstable or stable, it is essential that when assessing suffusion the actual gradations of the soil are considered, not averaged values. It will also be necessary to have a number of test data from the zone of the dam or the foundation which cover the range of soil gradations present.

8.5.5.3 *Allow for the effects of segregation when assessing suffusion*

Soils which are subject to suffusion are also subject to segregation in stockpiles prior to placement and during placement on the embankment. Segregation is also a problem for broadly graded soils such as those in Figure 8.48 and in filters.

Characteristics which make it more likely that silt-sand-gravel soils will segregate, are described in Section 9.2.8.2.

Some information on segregation may be available from particle size distributions taken during construction, from photographs of placement during construction, or from samples taken from test pits in the embankment.

Segregation is quite critical because it may result in layers of coarser soil which are internally unstable whereas the average gradation is not.

8.6 CONTACT EROSION

8.6.1 General description of contact erosion

Contact erosion occurs where a coarse soil such as gravel is in contact with a fine soil, and flow parallel to the contact in the coarse soil erodes the fine soil as shown in Figure 8.57. For example flow through gravel alluvium in the foundations of dam or dike may erode the base of an overlying silt layer, or erosion of the finer layers of soil in a core may occur into a coarse gravelly layer formed by segregation during construction. Figure 8.58 shows examples.

Figure 8.59 shows some of the potential consequences of contact erosion.

8.6.2 Methods for predicting initiation and progression of contact erosion

Two conditions are needed for initiation of contact erosion:

Geometrical condition: pores of the coarse layer have to be sufficiently large to allow particles to pass through. Some authors proposed criteria in the specific context of contact erosion but classic filter criteria could also be used.

Hydraulic condition: Flow velocity has to be sufficient to detach the particles and also to transport them.

Different authors who have studied contact erosion proposed expressions for the hydraulic conditions giving detachment and the transport of particles. These have been established based on experimental results. These authors noticed that close to the

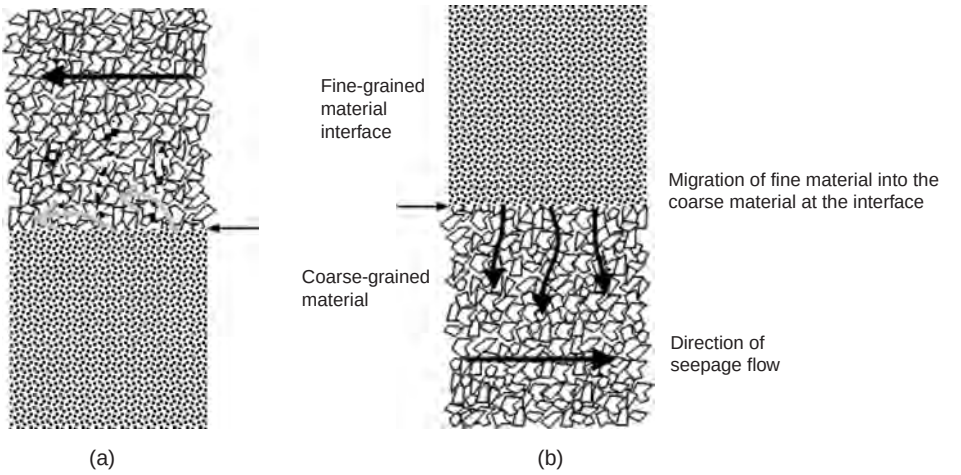


Figure 8.57 Diagram showing soil contact erosion (a) Fine soil below coarse soil; (b) Fine soil above coarse soil (ICOLD, 2013).

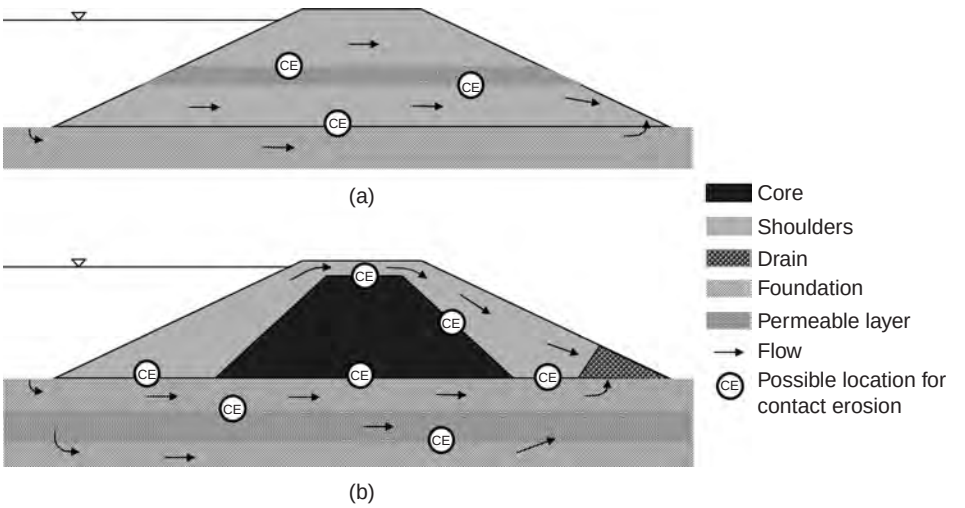


Figure 8.58 Examples of where contact erosion may occur. (a) Homogeneous dam with layered fill due to segregation during construction and a coarse foundation soil. (b) Zoned dam with potential for contact erosion at high reservoir levels above the core and for erosion into coarse layers in the foundation (Beguin *et al*., 2009).

geometrical limit, the hydraulic loading for erosion to initiate is increasing. They noted a transition zone of combined influence where the hydraulic loading needed to initiate contact erosion is higher than in the domain of pure hydraulic influence as shown in Table 8.15.

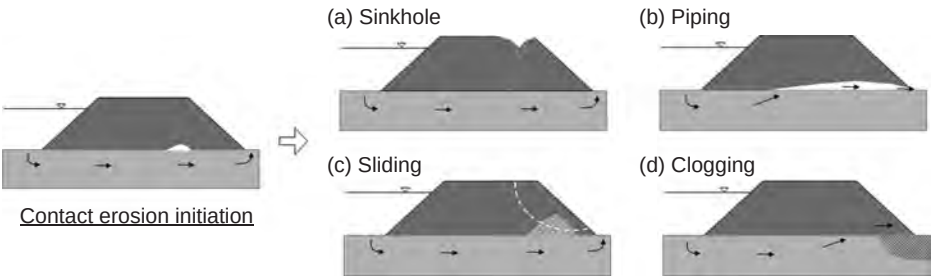


Figure 8.59 Potential consequences of contact erosion. Black arrows indicate a groundwater flow through a more permeable layer (light grey) under a less permeable dam (dark grey). (a) sinkhole daylight; (b) beginning of backward erosion piping; (c) creation of a weaker zone initiating instability; (d) clogging of the permeable layer and increase of pore water pressure (Beguin *et al.*, 2010).

Table 8.15 Domain of geometrical and hydraulic influence (ICOLD, 2013, courtesy of R. Beguin, from Brauns, 1985, Wörman, 1992 and Den Adel, 1994).

	Grading ratio D_{15}/d_{85} →				
Brauns (1985) soil with $n = 0.4$	Geometrical condition	7.5	Geometrical and Hydraulic condition	25	Hydraulic condition
Wörman (1992) soil with $D_{15} = 0.88D_H$				14.6	
Den Adel (1994) soil with $d_{85} = d_{50}/0.9$		8.1		11.7	

Note. D_{15} is the particle size of the coarser soil for which 15% is finer; d_{85} is the particle size of the finer soil for which 85% is finer. n is porosity.

Table 8.15 is derived from Brauns (1985), Wörman and Olafsdottir (1992) and Den Adel *et al.* (1994). They found that for D_{15}/d_{85} ratios less than in the third column, there is geometrical filtration whatever the hydraulic loading, so contact erosion could not occur. For D_{15}/d_{85} ratios more than in the fifth column hydraulic loading controls erosion and there is no filtration effect. In between these two limits both geometric and hydraulic factors control erosion.

It is notable that these ratios are close to the continuing erosion $D_{15}/d_{85} < 9$ proposed by Foster (1999), Foster and Fell (1999a, 2001). So it seems that contact erosion will only occur where the coarse strata can provide no filtering capability against the finer soil.

The second boundary between "Geometrical and Hydraulic condition" and "Hydraulic condition", defines if the coarse layer grading has, or does not have, an influence on the hydraulic criteria for erosion initiation.

For one particular fine soil, the "critical" gradient, which corresponds to the gradient in the coarse layer parallel to the contact at which the erosion initiates, can vary by one order of magnitude, depending of the permeability of the coarse layer. However, in the same tests, the "critical" Darcy velocity (= ki, where k = hydraulic conductivity, and i = seepage gradient) for erosion initiation does not significantly depend on the coarse layer permeability, and is only related to the fine soil resistance to erosion.

Table 8.16 Likelihood of a soil being able to support a roof to an erosion pipe (Fell *et al.*, 2008).

Soil Classification	*Percentage Fines*	*Plasticity of Fines*	*Moisture Condition*	*Likelihood of Supporting a Roof*
Clays, sandy clays (CL, CH, CL-CH)	>50%	Plastic	Moist or saturated	1.0
ML or MH	>50%	Plastic or non-plastic	Moist or saturated	1.0
Sandy clays, Gravely clays, (SC, GC)	15%–50%	Plastic	Moist or Saturated	1.0
Silty sands, Silty gravels, Silty sandy gravel (SM, GM)	>15%	Non plastic	Moist Saturated	0.7 to 1.0 0.5 to 1.0
Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)	5% to 15%	Plastic	Moist Saturated	0.5 to 1.0 0.2 to 0.5
Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)	5% to 15%	Non plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils, (SP, SW, GP, GW)	<5%	Non plastic Plastic	Moist and saturated Moist and saturated	0.0001 0.001 to 0.01

Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e. not rolled), and upper bound for well compacted materials.
(2) Cemented materials give higher probabilities than indicated in the table. If soils are cemented, use the category that best describes the particular situation.

Therefore the Darcy velocity has been chosen by the majority of authors as a good indicator of the hydraulic loading.

The hydraulic conditions for contact erosion depend of the configuration considered.

8.6.2.1 *Non plastic sand below a coarse soil layer*

This configuration has been widely studied (Istomina, 1957; Pravedny, 1966; Brauns, 1985; Bezuijen *et al.*, 1987).

Beguin (2011) indicates that in the case of sand erosion particles are being mainly transported as bed load and the various authors concluded that classical river erosion criteria can be empirically adapted to the case of contact erosion. In consequence, they proposed methods based on Shields criterion. Experimental results from a number of authors are shown in Figure 8.60 with some existing criteria:

Experimental results range between 0.01 m/s and 1 m/s for the critical Darcy velocity U_{crit} (m/sec) and a minimum seems to occur for particles of diameter 1.0E−04 m (100 μm). Beguin (2011) indicates that Brauns (1985) is the simplest formula to use and gives a good approximation for sand:

$$U = 0.70 n_D \sqrt{\left(\frac{\rho_s - \rho_w}{\rho_w}\right) g d_{50}} \tag{8.10}$$

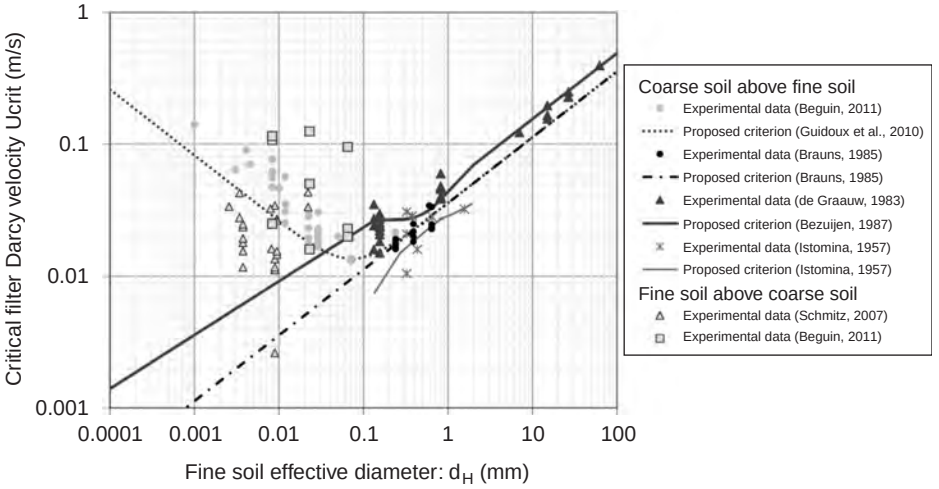


Figure 8.60 Critical velocities for contact erosion of soil above and below gravel (ICOLD, 2013, courtesy of R. Beguin).

in which n_D the porosity of the gravel layer, ρ_s (kg/m^3) the density of the sand particles, ρ_w (kg/m^3) the water density, and d_{50} (m) the median diameter of sand grading curve. This applies for $n_D d_{15}/d_{85} > 10$.

To predict the rate of erosion evolution, three models of sand erosion have been developed which allow estimation of the amount of transported sand as a function of the hydraulic loading (Wörman and Olafsdottir, 1992; Den Adel *et al.*, 1994; Scheuermann *et al.*, 2002). However these models have been validated only for the soils tested by the individual author and they have to be used with caution.

8.6.2.2 *Non plastic silt and clay (particles <75μm) below a coarse layer*

Guidoux *et al.* (2010) carried out some experimental tests of contact erosion with silt and clay. They adapted Brauns (1985) law with an empirical parameter to take into account the adhesive forces. To consider widely or gap graded soils, they also proposed to use the effective diameter dH of the fine soil instead of the d_{50}. The effective diameter d_H (m) is defined by:

$$d_H = \left(\sum_{j=1}^{m} \frac{F_j}{d_j} \right)^{-1}, \tag{8.11}$$

where F_j is the mass of the fraction of diameter, d_j in the grading curve of the soil.

This diameter conserves the specific surface of the initial grain size distribution.

Guidoux *et al.* (2010) proposed that the critical Darcy velocity U_{crit} can be determined from:

$$U_{crit} = 0.70.n_D\sqrt{\left(\frac{\rho_s - \rho_w}{\rho_w}\right) g d_H \left(1 + \frac{\beta}{d_H^2}\right)} \tag{8.12}$$

with β an empirical parameter $= 5.3 \times 10^{-9}$ m^2 according to the results of Guidoux *et al.* (2010). The other variables are as for Brauns (1985) above.

The experimental data in Figure 8.60 is very widely scattered (given it is a log-log plot). The Guidoux *et al.* (2010) equation appears to be at the lower boundary of the data. The data includes plastic soils and as discussed in Section 8.3.4 the mechanics of erosion of these soils are not controlled by particle size, so the scatter is not surprising.

8.6.2.3 *Non-plastic silt above a coarse soil layer*

In this configuration experimental data is limited.

Schmitz (2007) carried out experimental tests of erosion of silt layers above coarse layers. Except for one value, the critical Darcy velocities measured are of the same order of magnitude as in the other configuration, between 0.01 m/s to 0.1 m/s (Figure 8.60).

Beguin (2011) carried out six tests on silt embankments over gravel. The results are also shown in Figure 8.60. The critical velocities were more scattered than those for the gravel above silt configuration, but of a similar order of magnitude. He indicated that it seems the initiation of erosion is dependent on the transport of particles, not by detachment.

The data scatter is so large it appears that all that can be said is that for these soils the critical Darcy velocity is between 0.01 and 0.1 m/s.

8.6.2.4 *General comment*

Given the approximate nature of the prediction methods for contact erosion involving erosion of silt soils, and the fact that erosion of clay soils is not related to particle size, in these cases laboratory experiments modelling the field situation would seem to be necessary.

8.6.3 Contact erosion or scour of the dam core into open joints in rock in the foundation

A form of contact erosion (also called scour) may occur where the core of a dam is placed on open jointed rock. The flow of water in the cracks may erode the core at the contact. Figure 8.61 shows the process.

The method of assessing this is:

- Assess the likelihood of a continuous pathway for erosion at the core-foundation contact (P_{path}). This should be assessed from the geology of the foundation, mapping and photographs taken during construction, grouting records, the depth of general foundation and cut off excavation, treatment carried out on the walls and floor of the cut off trench (e.g. shotcrete, slush concrete, dental concrete). In many cases it will become apparent at this stage that there is little or no likelihood of

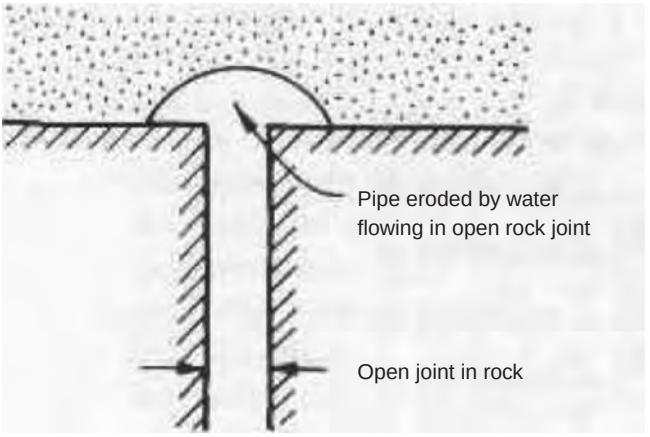


Figure 8.61 Contact erosion (or scour) of the core of a dam into an open joint in the foundation.

such features being present, or the defects are not continuous and the probability of internal erosion into the foundation may be assessed as negligible.

- Assess the likelihood of erosion of the core material at the core-foundation contact (P_{ic}) using the method for erosion in a crack in the core as described in Section 8.3.4. The hydraulic gradient used in the assessment should be based on the estimated seepage gradient on the core-foundation contact. Assume that the hydraulic shear stresses imposed on the core by the water flowing in the open joints is equivalent to those for an equivalent crack width.
- The probability of internal erosion by contact erosion or scour $= P_{path} \times P_{ic}$. That is, for scour to occur both conditions must be met.

8.7 CONTINUATION AND FILTER ACTION

See Chapter 9.

8.8 PROGRESSION OF EROSION

8.8.1 General description

Progression is the phase of internal erosion where:

- For concentrated leak erosion, the erosion in the crack or concentrated leak leads to development of a pipe.
- For backward erosion the erosion process extends upstream from the point of initiation, and a network of small erosion channels forms beneath the soil or

embankment providing the roof to the erosion pipes. If these small erosion channels reach the reservoir or river, then a pipe forms.
- For suffusion, some of the finer fraction is eroded leaving the coarse matrix of the soil. No pipe is formed but the permeability of the soil may be increased significantly.
- For contact erosion the erosion of the finer soil into the coarser soil continues. This may in particular cases lead to development of a pipe in the finer soil.

Progression of backward erosion, suffusion and contact erosion is described above. The following describes how progression is assessed for concentrated leak erosion.

8.8.2 Overall approach for assessing progression for concentrated leak erosion

The likelihood of concentrated leak erosion progressing depends on whether:

- The soil will "hold a roof" over a pipe.
- "Crack filling" action will not stop the erosion process.
- Flow in the developing pipe will not be restricted by an upstream zone or hydraulic losses in upstream and downstream zones (or for example a concrete face slab).

It should be noted that in the absence of crack filling and flow limitation, or lowering of the reservoir to reduce the gradient of flow in the developing pipe, erosion will progress. This is because the hydraulic shear stresses in the pipe increase as the pipe enlarges.

8.8.3 Assessing whether the soil will hold a roof to a developing pipe

For internal erosion and piping through the dam or piping from the embankment into a rock foundation, the core must be capable of holding the roof of a pipe.

Based on case studies (Foster 1999), (Foster and Fell 1999a), the most important factors are:

- The fines content of the soil (% passing 0.075 mm) – $\geq$15% fines likely to be able to hold a roof regardless of whether the fines were non plastic or plastic.
- Whether the soil was partially saturated or saturated.

Other factors which were considered to be likely to have an influence, were degree of compaction (loose soil would be less likely to support a roof to a pipe than dense) and reservoir operation (cyclic reservoir levels were more likely to cause collapse than steady).

Fell *et al.* (2008) developed Table 8.17 based on these case data and taking into account of results of testing by Park (2005) which showed sandy gravels with 5 to 15% non-plastic fines collapsed quickly when saturated. Sandy gravels with 5% cohesive fines collapsed after some time, but very slowly with 15% cohesive fines.

Table 8.17 Estimation of the likelihood of crack filling action not stopping pipe enlargement – internal erosion through the embankment (Fell *et al.*, 2008).

Embankment Zoning	*Upstream Granular Zone*	*Downstream Filter or Transition or other granular material*	*Likelihood of Piping Progressing – Crack Filling Action Not Effective*
Homogeneous, earthfill with toe drain, earthfill with horizontal drain, concrete face earthfill, puddle core earthfill, earthfill with core wall, hydraulic fill	None except for rip rap and filters under these	None or none effective	1.0
Earthfill with vertical and horizontal drain, zoned earthfill	None	Present	1.0
Central and sloping core earth and rockfill (or gravel shoulders)	Present	Present	0.1 to 0.9 If the core is well graded and has fine to coarse sand sizes (0.075 – 4.75 mm) already present[(1)] 0.01 to 0.1 If the core is deficient in sand sized particles and washed in sand material aids in sealing the downstream zone[(2)]

Notes: [(1)] Crack filling is more likely to stop pipe enlargement when the core zone is deficient in sand size particles and these particles can be provided by washing in from the upstream zone. This aids in sealing of the downstream filter zone. If the core is well graded and has sand sizes present, then the potential benefits of crack filling are less as the sand size particles are already present.

[(2)] Probability dependent on compatibility of particle sizes of granular soils upstream of the core and in the downstream filter transition.

8.8.4 Assessing whether crack filling action will occur

8.8.4.1 *Internal erosion in the embankment*

For crack filling to occur requires a granular zone upstream of the core, with particles of a size which can be transported by the water flowing into the crack or pipe, and a downstream filter transition zone or rockfill which is sufficiently fine, to act as a filter to these particles. Case studies where crack filling was evident include Matahina Dam (Gillon, 2007), Swedish dams (Nilsson, 2007a, b) and others in Foster and Fell (1999b). These are typified by sinkholes appearing above the upstream filter zone which is taken as evidence of material being washed into the pipe.

Table 8.17 shows the likelihood that crack filling will not be effective. It shows that the greatest benefit is in cases where there is poor filter compatibility between the core and downstream filter due to a lack of sand size particles in the core. In these cases, the likelihood of continuation may be high, but the washed in sand sized materials may be capable of filtering against the downstream filter zone and this reduces the potential for the pipe enlarging and internal erosion progressing. There is less benefit where the

materials that are washed in, are of similar sizes to those already in the core. Hence the likelihood of crack filling not being effective in Table 8.17 are higher for a well-graded core material compared to those for a core which is deficient in sand sizes. There is very little benefit where there is no downstream filter/transition zone present for the washed in materials to self-filter against.

8.8.4.2 *Internal erosion through the foundation*

The potential for crack filling action for internal erosion in the foundation depends on the zoning of the embankment and the geological features in the foundation.

If the eroding foundation soil layer is located immediately below the embankment, and the embankment forms the roof of the pipe, then evaluate the likelihood for crack filling action as for erosion in the dam using Table 8.17.

If the eroding foundation soil layer is located further below the embankment, then consider the potential for the overlying soil layers to wash into the developing pipe. There needs to be a filtering material at the downstream end of the flow path for crack filling action to be effective. The filtering material may be a naturally occurring layer in the foundation or the embankment filter.

8.8.4.3 *Internal erosion of the embankment into or at the foundation*

The issues are the same as for internal erosion through the embankment. Estimate the likelihood of crack filling action not stopping pipe enlargement using Table 8.17.

8.8.5 Assessing whether upstream flow limitation will occur

Upstream flow limitation may occur where there is a relatively fine-grained granular material (fine rock fill, or sandy gravels) upstream of the core, or where there is a concrete face slab or concrete core wall.

Fell *et al.* (2008) gives guidance on the assessment of the likelihood of upstream flow limitation being effective.

Both upstream and downstream zones can generate such high hydraulic losses according to the permeability of the filter such that the hydraulic stresses in the crack passing through the core are not sufficient for erosion of the core material to progress and the enlargement of the pipe stops.

Methods for assessing this are given an Appendix A.

8.8.6 Assessing the rate of development of the pipe

From the Hole Erosion Index of the eroding soil and average hydraulic gradient along the pipe it is possible to a estimate approximately the rate at which a piping hole will enlarge. This is important to know because it is a significant factor in assessing the likelihood of successful intervention to stop the piping process. The results are summarised in Figure 8.62 assuming (a) Unrestricted potential for erosion (i.e. no flow limitation, continuing erosion condition); (b) Initial pipe diameter of 25 mm; (c) Zero critical shear stress which is conservative, particular for $I_{HET} > 3.5$; (d) Reservoir

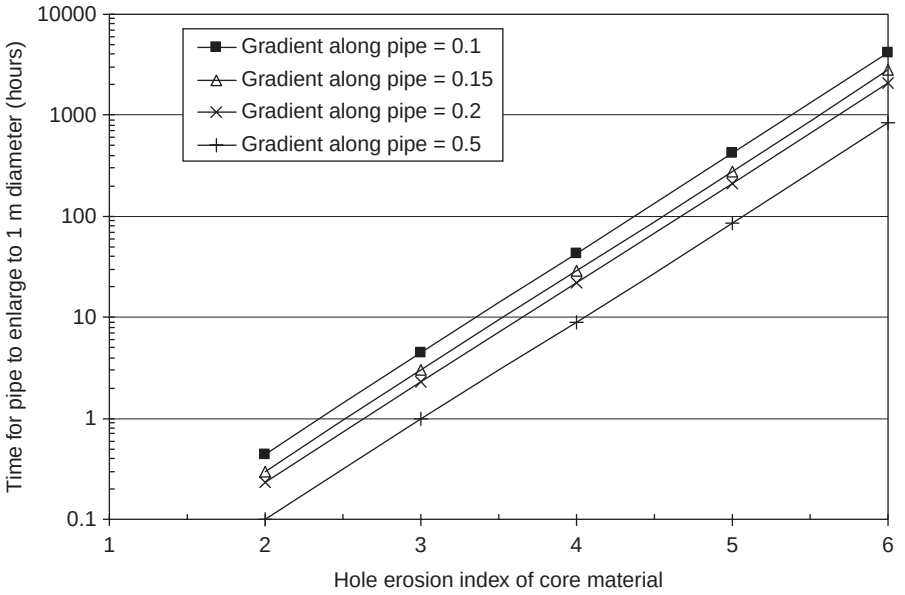


Figure 8.62 Approximate time for pipe to enlarge from 25 mm to 1 m diameter. The time to erode to 2 m diameter is about 20% greater.

level remains constant. It will be seen from Figure 8.62 that the time for erosion to progress is very dependent on the soil erosion properties. Caution needs to be taken in extrapolating the rate of erosion for large pipe diameters from I_{HET} data as the mechanism of particle detachment and erosion can change to larger clumps of soil detaching from the roof as the pipe enlarges.

The rates of development of the pipe are consistent with case studies such as Teton Dam shown in Figure 8.63 which had core material with an erosion rate index I_{HET} of 2 to 3, and the method for assessing the time to progress from the first signs of a concentrated leak to breach as described in Fell *et al.* (2001, 2003) and Section 8.9.2.

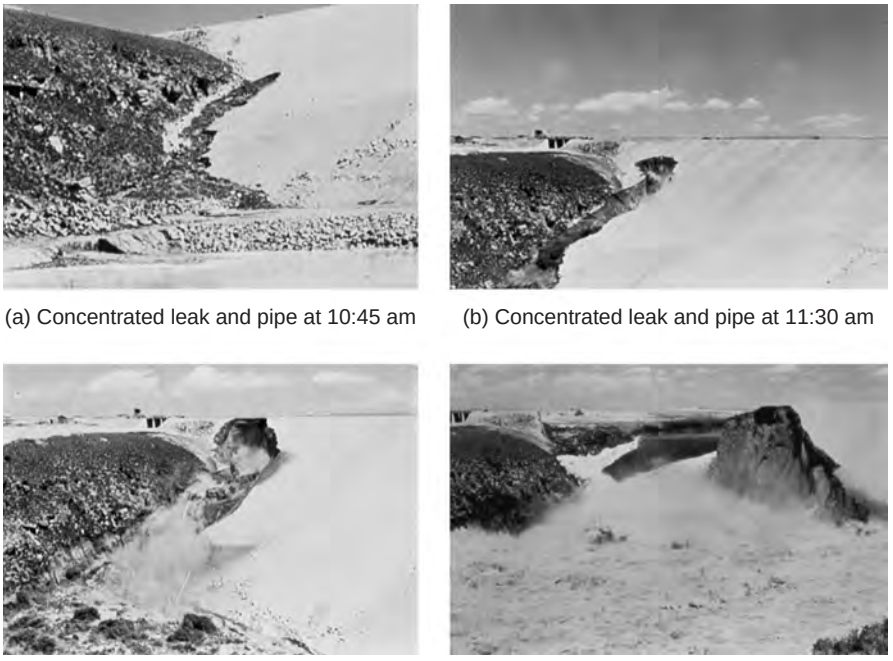
8.9 DETECTION OF INTERNAL EROSION AND PIPING

8.9.1 General principles

The likelihood that a particular failure path can be detected, and if so, whether it is possible to intervene (e.g. by lowering the reservoir level) or carry out repairs to prevent the dam breaching is usually best considered as two questions:

1. Will this failure path be detected?
2. Will intervention and repair be possible?

A probability is assigned to each of these questions. The overall probability of detection, intervention and repair is the product of these two probabilities.



(a) Concentrated leak and pipe at 10:45 am (b) Concentrated leak and pipe at 11:30 am

(c) Breach nearly developed at 11:55 am (d) Dam breached early afternoon

Figure 8.63 Photographs of development of the breach of Teton Dam in 1976.

The likelihood of detection and successful intervention and repair is dependent on a number of factors including:

a) The category of internal erosion and piping, i.e. internal erosion in the embankment, the foundation or embankment to foundation.
b) The mechanism of initiation of internal erosion – erosion in a crack, suffusion or backward erosion.
c) The breach mode – gross enlargement of a pipe, instability of the downstream slope, unraveling or sloughing of the downstream slope, settlement of the foundation, sinkhole development.
d) The nature of and the geometry of the materials in the foundation.
e) The zoning of the embankment, and the materials in the embankment.
f) The reservoir level at the time of the piping incident, and how rapidly it can be drawn down.
g) The type and frequency of monitoring and surveillance at the dam and the training of the staff to recognize a developing internal erosion and piping incident.
h) The ability to get trained personnel out to the site in the event of a piping incident.
i) The ability of those responsible to be able to direct emergency release of the reservoir
j) The availability of materials and equipment to intervene and carry out repair works.

Table 8.18 A method for the approximation estimation of the time for progression of piping and development of a breach, for breach by gross enlargement and slope instability linked to development of a pipe (Fell *et al.*, 2001, 2003).

Factors Influencing the Time for Progression and Breach					
Ability to Support a Roof From Table 8.15	*Rate of Erosion From Table 8.16*	*Upstream Flow Limiter*	*Breach Time From Table 8.17*	*Approximate Likely Time-Qualitative*	*Approximate Likely Time*
Yes	R or VR	No	VR or R-VR	Very Rapid	<3 hours
Yes	R	No	R	Very Rapid to Rapid	3–12 hours
Yes	R-M	No	VR	Rapid	12–24 hours
Yes	R	No	R-M		
Yes	R	No	M or S	Rapid to Medium	1–2 days
Yes	R or R-M	No	M or M-S		
Yes	M or R-M	Yes	R or R-M		
Yes	M or R-M	No	S	Medium	2-7 days
Yes	R-M or M	Yes	S		
Yes	M	Yes or No	S	Slow	Weeks – even months or years

Note: VR = Very Rapid; R = Rapid; M = Medium; S = Slow.

8.9.2 Some information on the rate of internal erosion and piping

The likelihood of detection and successful intervention or repair depends on the time from when the internal erosion process may be detected to when breach begins.

Fell *et al.* (2001, 2003) studied case histories of failures and accidents for piping in the embankment, foundation, and embankment to foundation. Based on the case histories and an understanding of the physical processes they provided guidance on the time for progression beyond when a concentrated leak is first observed and development of a breach. Tables 8.18 to 8.21 are based on that study.

Table 8.18 should be used to estimate the approximate likely time to dam failure after a concentrated leak is first observed. Tables 8.19, 8.20, 8.21 are used in Table 8.18 working from left to right.

Table 8.19 replaces an original table to assess the likely rate of erosion of the core of the embankment or the soil in the foundation.

In these tables the terms for rates are defined as shown in Table 8.21. Dual descriptors are used to describe intermediate terms e.g. very rapid – rapid for 6 hours. The terms are applied to part (e.g. progression) or the whole process.

Most of the cases studied were for breach by gross enlargement, so the method is applicable to cases where the mechanism is gross enlargement. It is considered to be reasonably applicable to cases where the final breach is by slope instability, following development of a pipe. It will probably underestimate the time for breach by sloughing. Sloughing is a slowly developing breach mode which should take days or weeks to lead to breach.

Table 8.19 Rate of Erosion of the core or soil in the foundation.

Soil Classification	*Best Estimate Erosion Rate Index (I_{HET})*	*Time for erosion in the core of the embankment or in the foundation*	
		Gradient along pipe 0.2	*Gradient along pipe 0.5*
SM with <30% fines	<2	Very Rapid	Very Rapid
SM with >30% fines	2 to 3	Very Rapid	Very Rapid
SC with <30% fines	2 to 3	Very Rapid	Very Rapid
SC with >40% fines	3	Rapid	Very Rapid
ML	2 to 3	Very Rapid to Rapid	Very Rapid
CL-ML	3	Rapid	Very Rapid
CL	3 to 4	Rapid	Very Rapid to Rapid
CL-CH	4	Rapid	Rapid
MH	3 to 4	Rapid	Very Rapid to Rapid
CH with Liquid Limit <65%	4	Rapid to Medium	Rapid
CH with Liquid Limit >65%	5	Medium to Slow	Medium

Table 8.20 Influence of the material in the downstream zone of the embankment on the likely time for development of a breach.

Material Description	*Likely Breach Time*
Coarse grained rockfill	Slow–medium
Soil of high plasticity (plasticity index >50%) and high clay size content including clayey gravels	Medium–rapid
Soil of low plasticity (plasticity index <35%) and low clay size content, all poorly compacted soils, silty sandy gravels	Rapid–very rapid
Sand, silty sand, silt	Very rapid

Table 8.21 Qualitative terms for times of development of internal erosion, piping and breach (Fell *et al.*, 2001, 2003).

Qualitative Term	*Equivalent Time*
Slow (S)	Weeks or months, even years
Medium (M)	Days or weeks
Rapid (R)	Hours (>12 hours) or days
Very Rapid (VR)	<3 hours

Breach by sinkhole development is potentially a rapid process in the final stages when the sinkhole emerges into the reservoir. Breach would be expected to occur in a small number of hours; there is not enough case data to support a more refined estimate.

Table 8.18 is used by assigning the values to the first four columns and selecting the likely time for progression and breach which best fits the data.

Note that the dispersivity of the soil does not significantly affect the rate of erosion so is not listed as a factor in Table 8.19. For a homogeneous dam the whole of the embankment is the same soil, so in Table 8.19 the soil is considered as the core and in Table 8.20 as the downstream zone.

Fell *et al.* (2001, 2003) show that the method gives a reasonable estimate of the time for progression beyond where a concentrated leak is observed and breach and the times are acceptably accurate for the purpose here which is to assess the likelihood of detection, intervention and repair. Fell *et al.* (2001, 2003) and the authors caution however, against over-reliance of these figures for life loss estimates where the estimates are sensitive to the assumed warning times. The times estimated in Table 8.18 are only approximate and hidden or unknown details within a dam or its foundation may give shorter or longer times.

8.9.3 The likelihood of detection and intervention

Detection may be possible in the continuation or early progression phase, or more likely, in the advanced stages of progression and breach formation. Detection is likely to be by:

1. Observation of increased seepage out of the downstream face of the embankment or in the foundation. This may be by visual observation, or by seepage measurement or more sophisticated methods such as thermal monitoring of the foundation or the downstream slope.
2. Measured higher pore pressures in the foundation and/or embankment.
3. Settlements, deformation and cracking in the embankment or area downstream of the dam.

Whether detection is likely depends on:

1. The rate at which the internal erosion and piping and associated processes, such as instability of the downstream face, occurs.
2. The frequency of inspections, and measurement of monitoring equipment.
3. The dam zoning and the location of the concentrated leak and whether the leak will be visible to those doing the inspection.

For example if a process may go from initiation or first presence of a concentrated leak to breach in say 6 hours, and the dam is only inspected or monitored weekly, it is very unlikely that a piping incident will be detected before breach occurs. However if the dam is visible by the general population, there is some chance the leak may be noticed nonetheless.

Detection early in the internal erosion process is usually difficult, particularly for erosion initiating along a crack or by backwards erosion because the amount of leakage is very small at the start. Fell *et al.* (2001, 2003) record that most piping incidents are first identified as a concentrated leak in the progression phase. Suffusion is more likely to be detected by piezometers because the process is slower to develop. The presence of conditions potentially leading to heave and backward erosion in the foundation

may also be detected by piezometers provided they are correctly positioned and read as reservoir levels rise.

Visual inspection is a vital tool in detecting internal erosion and piping. Whether it is successful would be dependent on the factors discussed above, but also on such practical issues as:

- Inspections are seldom practical at night, so there is 30% to 50% of the time (varying throughout the year) when detection will not be effective, particularly for rapidly developing piping mechanism. Many dams are not inspected on weekends, further reducing the likelihood of detection.
- Dense vegetation, runoff from rainfall, snow cover can all hide the presence of a concentrated leak. However it can be the case that melted snow is a good indicator of areas affected by seepage.
- For very long embankments, it is not practical to walk to inspect, so it is less likely small leaks are detected.

It is known that most internal erosion and piping failures occur at reservoir levels close to or above historic high, and the physical processes are driven by the reservoir water. Hence a good monitoring and surveillance program will have a greatly increased frequency of inspections and reading of critical instruments under such reservoir conditions.

Chapter 20 includes a review of methods of monitoring and detection and their applicability to internal erosion modes.

Appendix A includes guidance on how to assess the likelihood of detection of internal erosion and piping.

8.10 INTERVENTION AND REPAIR

Intervention and repair to prevent the progression of internal erosion and piping and breach can take several forms including:

(i) Drawing down the reservoir level using spillway gates or outlet valves.
(ii) Installing pressure relief wells in the foundation of the embankment.
(iii) Building reverse filters over "boils" or areas where eroding material is emerging from the foundation of the embankment.
(iv) Building a weighting berm to reduce the likelihood of heave, or slope instability, or unraveling.
(v) Dumping granular material (sand/gravel/rockfill) into the upstream side of sinkholes to try to block them.

More than one of these measures may be used together. Which could be applicable or feasible would depend on the particular circumstances of the dam.

Section 10.9 in Chapter 10 gives examples of intervention and repair.

It should be recognized that there may be reluctance on the part of the reservoir owner or operator to release water given the lost revenue that may result, or if release of reservoir water is likely to result in property damage and loss of life, for example if

levee banks downstream of the dam are likely to be overtopped by the flood resulting from release of the water.

Appendix A includes a method for assessing the likelihood of successful intervention and repair.

8.11 INITIATION OF BREACH

8.11.1 General principles

Breach of the dam occurs when there is uncontrolled release of the reservoir water through a section of the embankment in which the crest has dropped to below the reservoir level. This may occur by:

(i) Gross enlargement of a pipe in the embankment followed by settlement or collapse of the embankment into the pipe resulting in the crest dropping to below reservoir level and being overtopped.
(ii) Instability of the downstream slope of the embankment or embankment and foundation, resulting in settlement and overtopping of the crest.
(iii) Unravelling or sloughing of the downstream slope of the embankment resulting in settlement and overtopping of the crest.
(iv) Progressive development of a vertical sinkhole into a pipe in the core, resulting in settlement and overtopping of the crest.

Loss of the reservoir water through a pipe which does not collapse to breach the crest is not regarded as a breach in the dam since the rate of release of the water will be limited by the size of the pipe.

Figure 8.64 shows schematically these breach modes.

In most cases breach is almost certain if the filters or transition give continuing erosion conditions. The exceptions to this will be if the reservoir level drops below the inlet of the developing pipe before a breach mechanism has time to develop.

The likelihood of breach is often less than 1.0 if the filters or transition give some or excessive erosion conditions. For these cases, the seepage flows that develop through the dam or foundation will be limited. As discussed in Section 9.3.2 for:

- *Some Erosion* – the filtering materials initially allow erosion from the soil it is protecting, but it eventually seals up and stops erosion. Leakage flows due to piping can be up to 100 l/s, but are self healing.
- *Seals with Excessive Erosion* – the filter material allows erosion from the material it is protecting and in the process permits large increases in leakage flow (up to 1000 l/s), but the flows are self healing.

The likelihood of breach will depend on how well the dam can cope with the leakage flows that may occur.

For most dam types and failure modes the likelihood for breach development will be dominated by one or two of the potential breach mechanisms. Breach mechanisms will not necessarily be applicable to some dam zoning types or modes of piping and can be ignored.

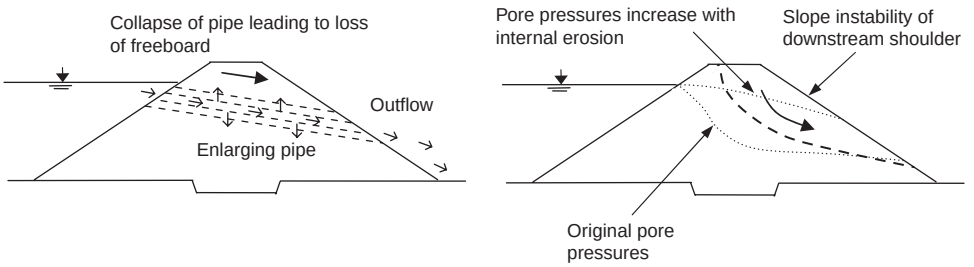

Collapse of pipe leading to loss of freeboard
Outflow
Enlarging pipe
Pore pressures increase with internal erosion
Slope instability of downstream shoulder
Original pore pressures
Breach by gross enlargement
Breach by slope instability

Figure 8.64a Potential breach (failure) phenomena-pipe enlargement and slope instability.

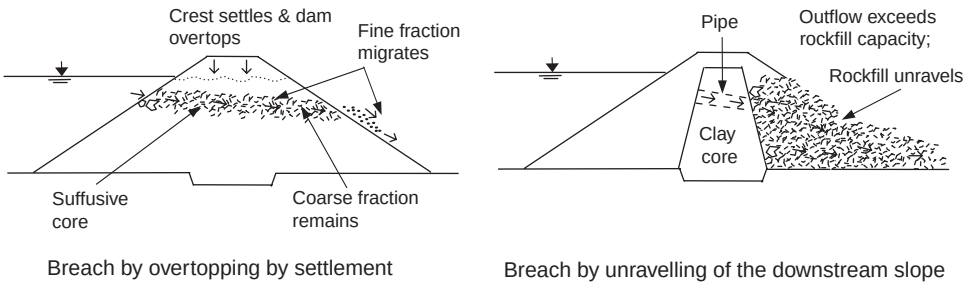


Figure 8.64b Potential breach (failure) phenomena-overtopping by settlement and unravelling of the downstream face (Fell and Fry, 2007).

Table 8.21 lists those breach mechanisms which should be considered in the assessment, depending on the dam zoning type and mode of internal erosion for internal erosion in the embankment due to a crack or poorly compacted zone. Table 8.22 also applies to internal erosion in a soil foundation. Figure 8.73 shows the dam zoning types.

Table 8.22 does not apply to failure modes involving open or in filled defects and solution features in rock foundations because the leakage flows may exceed the capacity of even free draining rockfill.

Internal erosion by the process of suffusion is very unlikely to lead to the formation of a pipe through the dam or its foundation and hence the probability of breach by gross enlargement where the mode of erosion is suffusion can be excluded. Breach by slope instability or sloughing/unraveling are usually the more critical mechanisms for suffusion, although the probabilities for breach are usually relatively low for this mode internal erosion.

Table 8.22 Screening of breach mechanisms for internal erosion through the embankment, internal erosion in soil foundations and from embankment into foundation (Fell *et al.*, 2008).

	Breach Mechanisms			
Dam Zoning Type	*Gross Enlargement*	*Slope Instability*	*Sloughing or Unravelling*	*Sinkhole Development*
Homogeneous earthfill	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
Earthfill with filters	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
Earthfill with rockfill toe	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
Zoned earthfill	Exclude, except if downstream fill can support a roof	✓	Exclude, except if downstream fill is cohesionless	✓
Zoned earthfill and rockfill	Exclude, except if downstream fill can support a roof	✓	✓*	✓
Central core earth and rockfill (or gravel shells)	Exclude, except if downstream fill can support a roof	Exclude, except if existing dam has marginal stability	✓*	✓
Concrete face earthfill	✓	✓*	Exclude, except if downstream fill is cohesionless	✓
Concrete face rockfill (including gravel fill)	Exclude	Exclude, except if dam is gravel or low permeability	✓*	Exclude
Puddle core earthfill	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
Earthfill with corewall	Exclude	✓*	Exclude, except if downstream fill is cohesionless	✓
Rockfill with corewall	Exclude	Exclude, except if existing dam has marginal stability	✓*	✓
Hydraulic fill	Exclude, except if downstream fill can support a roof	✓	✓*	✓

Key: ✓ Breach mechanism should be included for the probability estimate.
✓* Breach mechanism should be included for probability estimate, and usually is the more critical mechanism.

8.11.2 Breach by gross enlargement

For breach to occur by gross enlargement of a pipe; the pipe must stay open until it is so large that the settlement of the crest due to the pipe, or collapse of the embankment into the pipe lowers the crest to below the reservoir level.

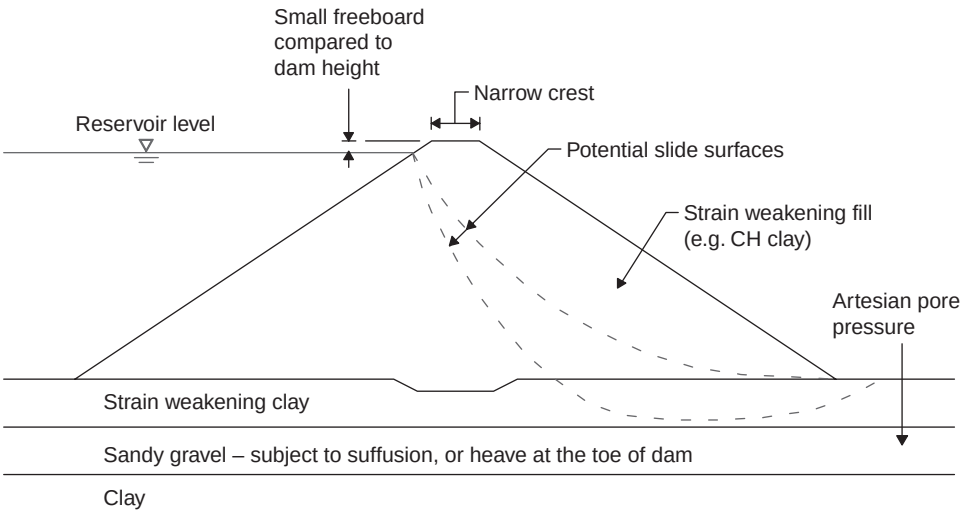


Figure 8.65 Conditions in which breach by slope instability is more likely.

If there is no intervention, the process can only stop if one or more of the following occurs:

- The hydraulic shear stresses in the pipe reach an equilibrium condition with the erosion resistance of the soil. This will not happen unless the reservoir level drops giving a lower gradient or the flow is limited by an upstream zone (because the hydraulic shear stress increases with pipe diameter for a constant gradient).
- The reservoir empties or falls below the entrance to the pipe before breach occurs.

This is because as the pipe enlarges the hydraulic shear stresses increase given the gradient remains the same. Section 8.8.6 describes the rate at which a pipe will develop related to the erosion rate index of the soil in the core.

8.11.3 Breach by slope instability

For breach to occur by instability of the downstream slope, the internal erosion and piping in the foundation must increase pore pressures in the embankment, or the embankment and foundation, so the factor of safety falls below 1.0, and the resulting sliding deformations must be such as to result in loss of freeboard so the reservoir overtops the dam crest.

Figure 8.65 shows a situation where breach by slope instability is more likely:

- There is small freeboard compared to the dam height and the dam crest is narrow.
- The soils in the embankment and foundation are strain-weakening, including loss of shear strength in the sandy gravel if a zero effective stress or heave condition is reached.

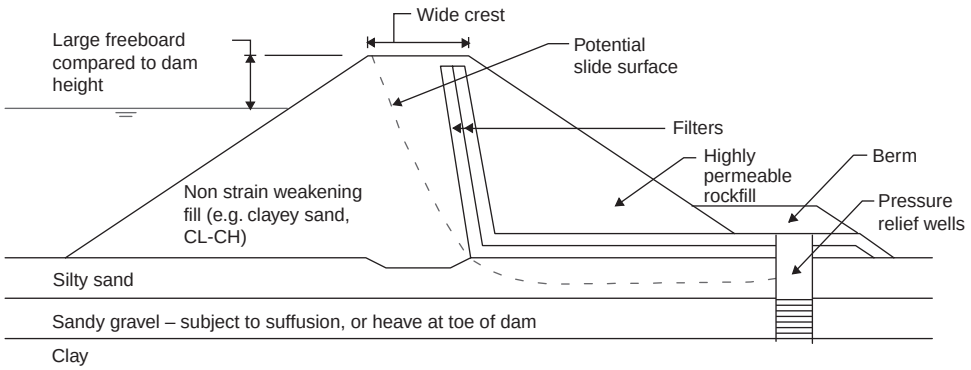


Figure 8.66 Conditions in which breach by slope instability is less likely.

- There is low discharge capacity in the downstream zone making a rise of pore pressure likely given a leak occurs.

Figure 8.66 shows conditions where breach by slope instability is less likely:

- There is a large freeboard compared to the dam height, and the dam crest is wide.
- The soils in the embankment and foundation are non-strain weakening.
- There is a large discharge capacity in the downstream rockfill zone making it unlikely pore pressures will rise given there is a leak

It should be noted that:

- Breach by slope instability may occur without a major pipe having formed. It may occur even if there has only been suffusion.
- The likelihood of breach is very dependent on the reservoir level and freeboard.
- The likelihood of breach is higher of the stability is marginal without the effects of internal erosion and piping.

There will be many situations lying between those shown in Figures 8.65 and 8.66, e.g. embankments with downstream zones of poor discharge capacity, but constructed of non-strain weakening soil.

In principle it may be possible to estimate the change in pore pressure resulting from the additional leakage resulting from internal erosion. In practice that will be difficult and it may be necessary to rely on judgement using the factors listed above.

It is not sufficient simply for the factor of safety to fall below 1.0, because with many soils, there is very little loss of strength on shearing beyond peak strength, so the resulting deformations may be small. Section 6.1 describes how to identify strain-weakening soils.

The deformations resulting from the factor of safety falling below 1.0 can be assessed using the Khalili *et al.* (1996) method described in Section 11.5.2 or by numerical methods.

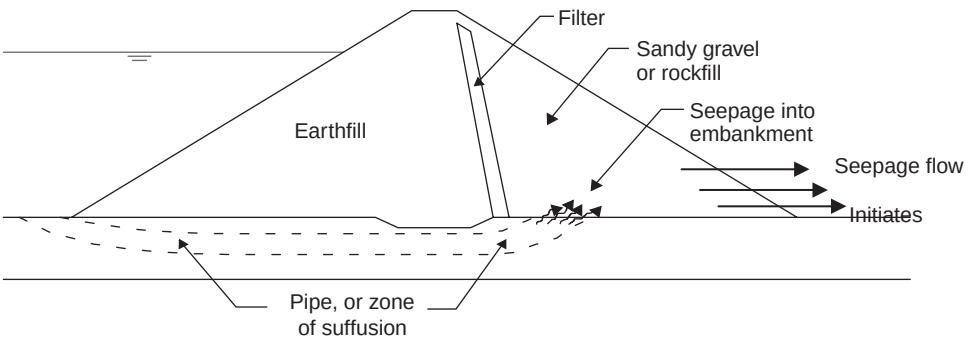


Figure 8.67 Breach by unraveling or sloughing of the downstream slope resulting from internal erosion and piping in the foundation.

8.11.4 Breach by unravelling or sloughing

For breach to occur by unravelling or sloughing of the downstream face of the embankment, the increased seepage due to internal erosion and piping in the foundation would have had to discharge into the downstream zone as shown in Figure 8.64(b) for piping through the embankment, and Figure 8.67 for piping in the foundation.

For sloughing to occur, the downstream face would have to be relatively steep, and the shoulder material a cohesionless soil, probably sandy gravel, or gravely sand, possibly with some silty fines. The process would have to be allowed to continue until it gradually eroded away the crest and allowed the reservoir to overtop the embankment.

Unravelling usually relates to the progressive removal of individual rocks by fairly large seepage flows flowing through the downstream rockfill, Foster and Fell (1999a) indicate that in the database they assembled, there was only one case of piping failure in an embankment with breach by unravelling. This was for a downstream zone of silty sand/sandy silt. Four other failures which were classified as slope instability failures as the breach mechanism (more deep seated sliding) all had 20–50% fines passing 0.075 mm, with the fines being non plastic or very low plasticity (plasticity index in the range 0 to 5%).

The potential for unravelling has often been assessed by the method of Sölvik (1981, 1991) which, as pointed out by EBL (2005), is the Olivier (1967) method, converted to metric units. It predicts the rock size d_s (m) to withstand unit discharge through the rockfill q_t (m^2/sec) for a downstream slope of S_0 (measured as V/H) by:

$$d_s = 1.46 q_t^{0.66} S_0^{0.78} \tag{8.13}$$

Based on model tests of flow through conditions including dams up to 6 m high, EBL (2005) showed that this method is quite conservative as shown in Figure 8.68. They point out that the Olivier method was based on tests of rockfill placed on sloping flumes.

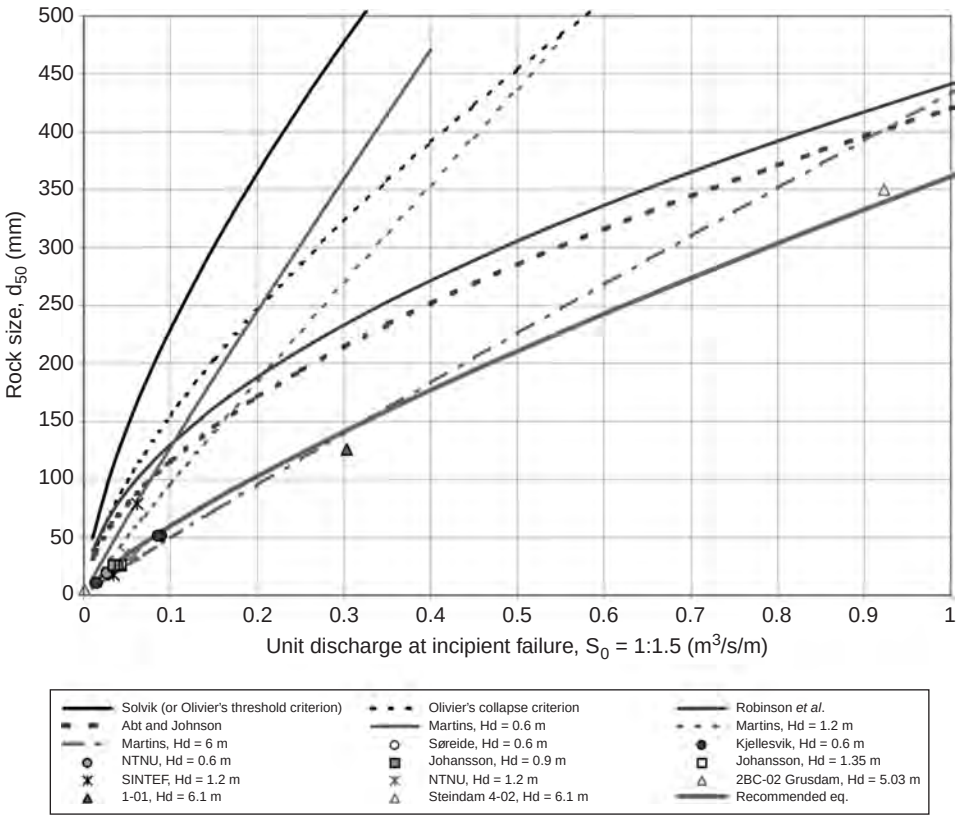

Rock size, d_{50} (mm)
Unit discharge at incipient failure, S_0 = 1:1.5 (m^3/s/m)
Solvik (or Olivier's threshold criterion)
Abt and Johnson
Martins, Hd = 6 m
NTNU, Hd = 0.6 m
SINTEF, Hd = 1.2 m
1-01, Hd = 6.1 m
Olivier's collapse criterion
Martins, Hd = 0.6 m
Søreide, Hd = 0.6 m
Johansson, Hd = 0.9 m
NTNU, Hd = 1.2 m
Steindam 4-02, Hd = 6.1 m
Robinson et al.
Martins, Hd = 1.2 m
Kjellesvik, Hd = 0.6 m
Johansson, Hd = 1.35 m
2BC-02 Grusdam, Hd = 5.03 m
Recommended eq.

Figure 8.68 Rock size at incipient failure for flow through rockfill (EBL, 2005).

They proposed that the best fit of their data was represented by:

$$d_{50} = 0.43 S_0^{0.43} q_t^{0.78} \tag{8.14}$$

where d_{50} (m) is the rockfill 50% passing size.

Nilsson (2013) indicates that the Norwegian dam safety regulator has required that a factor of safety be applied to this and design of berms to cope with through flow should satisfy:

$$d_{min} = 1.0\, S_0^{0.43} q_t^{0.78} \tag{8.15}$$

It is suggested that for assessing the likelihood of unravelling equation (8.14) be used. For design of berms for stabilizing flow through conditions, Equation 8.15 should be used. This is discussed further in Section 10.9.

When considering breach by unravelling, allowance should be made for the topography of the foundation of the dam as this is likely to result in concentration of flows into low points.

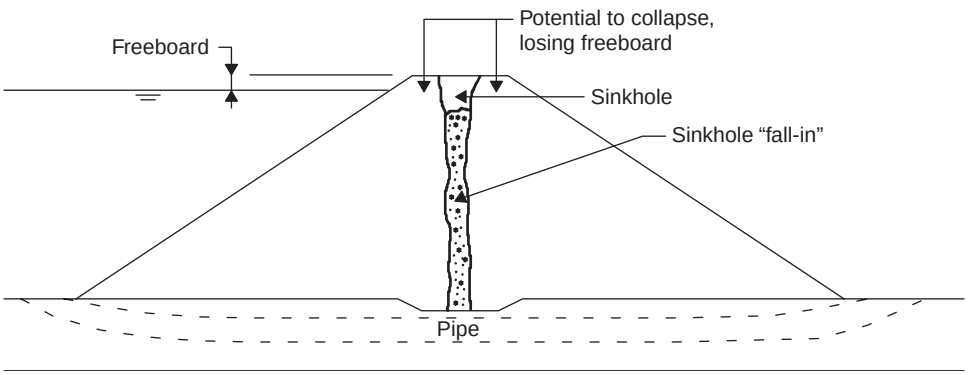


Figure 8.69 Development of a sinkhole eroding into a pipe in the foundation.

Table 8.23 Summary of depth of sinkholes observed in piping accidents (all piping modes) (Foster and Fell, 1999a, 2000).

Depth of Sinkhole (m)	*No. of Cases*	*% of Cases*
$<$1.0	10	22%
1.0–1.9	11	24%
2.0–2.9	7	16%
3.0–3.9	8	18%
4.0–4.9	3	7%
$\geq$5.0	6	13%

8.11.5 Breach by sinkhole development leading to loss of freeboard

Figure 8.69 shows the development of a sinkhole which may result in loss of freeboard.

Foster and Fell (1999, 2000a) indicate that for their database sinkhole and localized crest settlements were very common in piping incidents, (about 60% of incidents in the embankment, and 30% of incidents in the foundation) but have only led to three failures, for piping in the embankment and around conduits. This compares to 53 accidents, so only about 1 in 20 develop into a breach. Of these, two had been apparent for a long while and should have been fixed before failure. Most sinkholes are relatively shallow and only about half form on the crest.

Tables 8.23 and 8.24 summarize the data on sinkholes from all piping modes – piping in the embankment, foundation, and embankment to foundation.

The larger sinkholes were of a similar diameter to their depth.

The authors are not aware of any way of estimating the size or rate at which sinkholes might develop. It is necessary therefore to rely on this historic data to get a feel for what might happen. It should be remembered that the "depth" of sinkholes in Table 8.23 is the depth of the void at the top when the sinkhole was first observed and the actual sinkhole will mostly be greater depth.

Table 8.24 Summary of location of sinkholes observed in piping accident cases (all piping modes) (Foster and Fell, 1999a, 2000).

Depth of Sinkhole (m)	*No. of Cases*	*% of Cases*
On crest	25	47%
On upstream slope	17	32%
On downstream slope	11	21%

8.12 ASSESSMENT OF THE LIKELIHOOD OF INTERNAL EROSION AND PIPING IN EXISTING DAMS

8.12.1 General procedure

This section provides information on how the likelihood of failure of a dam by internal erosion can be determined. This may be done by "engineering judgement", qualitative analysis, and quantitative analysis.

The general procedure for estimation of the likelihood of failure a dam by internal erosion in the embankment, the foundations, and from embankment to foundations is:

(1) Assemble all the relevant information about the dam and its foundations as described in Section 8.12.2.
(2) Determine the loading conditions for the dam resulting from the reservoir and earthquakes as described in Section 8.12.3.
(3) Carry out a detailed Potential Failure Modes Analysis (PFMA) as described in Section 8.12.4.
(4) Screen out PFM which are not applicable to the dam or which will contribute a negligible amount to the likelihood of failure. Some guidance on how this may be done is given in Section 8.12.5. Screening may be done on the zoning of the dam and the foundation conditions, or a more detailed understanding of the factors influencing the initiation and continuation of erosion.
(5) For the remaining PFM use engineering judgement, qualitative or quantitative Risk analysis methods to estimate the likelihood of failure taking account the frequency of the applied reservoir and seismic loads. Which approach is used depends on the preference of the owner, dam engineer, and dam safety regulator. Some guidance on how this may be done is given in Section 8.12.5.

8.12.2 The importance of having complete and reliable information upon which to make the assessment of internal erosion

It is essential that the assessment of internal erosion of a dam be based on the best available information. It is best carried out in association with assessing other potential failure modes such as slope instability as there may be interactions between failure modes.

The data to be gathered includes the following.

8.12.2.1 *Geometric model*

The geometric model compiles the details of the internal geometry of the dam. Is it zoned, and what are the geometric details of the zones and the foundation contact of the fill within the various zones?

8.12.2.2 *Geological model of the foundation*

The geological model identifies the lithology and stratigraphy of the soil and rock, structural analysis (tectonic, position of discontinuities: joints, faults, etc.) and geological background (earthquake, geomorphology). The susceptibility to erosion of the soils overlying rock, and in some cases the erosion characteristics of weathered or soft rock and opening of joints, and the presence of features such as karst openings are the key factors.

The hydrogeology of the foundation should also be understood.

8.12.2.3 *Geotechnical model of the embankment and foundations*

The geomechanical model is the compilation of the mechanical properties required for internal erosion assessment and stability analysis, including soil classification and particle size, permeability, bulk density, and preferably data on the erosion properties of soils based on laboratory tests.

8.12.2.4 *Hydraulic or seepage model*

It is fundamental to understand the hydrodynamic situation in the dam and its foundations. The routes of seepage paths through and beneath the dam can be determined from piezometric readings, Seepage and hydraulic gradients through the dam and its foundations in normal operating and extreme circumstances should be considered.

8.12.2.5 *Stress state in the dam and its foundation*

This may be required for detailed assessments the likelihood and depth of cracking and hydraulic fracture.

8.12.2.6 *General comments*

To the maximum extent practicable, existing data from design and construction records, performance history including monitoring and surveillance data, and post-construction investigations need to be reviewed. It should not be assumed that the dam was constructed to the geometry shown in design drawings, and even "as constructed" drawings can be inaccurate. This is particularly so in the upper parts of the dam which are critical for concentrated leak erosion, and it may be necessary to carry out site investigations to confirm the geometry and properties in some cases.

At preliminary stages of an assessment, existing data supplemented by engineering judgment may provide a sufficient basis for evaluation.

However if the initial assessment indicates a high likelihood of internal erosion, particularly for high consequence of failure dams additional field investigations,

laboratory testing, and analyses may be warranted. An example is the particle size distributions of the core and the filter or transition. If this information is not available there is a chance the assessment of internal erosion will be completely wrong and misleading.

A site visit to the dam has been found to provide valuable input into the assessment process and in the authors view is an essential part of any assessment. The site visit allows a better understanding of the layout of the dam, details of the design and performance, an understanding of the geologic and topographic setting and helps in the process of identifying potential failure paths.

8.12.3 Loading conditions

The likelihood of failure of a dam by internal erosion and piping is dependent on the frequency (annual probability) of the load condition which may lead to potential cracks or flaws in the core being reached by the reservoir level; or the earthquake loading which may lead to cracks below the reservoir level; or the reservoir level at which critical gradients at which backward erosion piping, suffusion or contact erosion may initiate and progress. Even where the dam safety is being assessed by qualitative rather than quantitative risk assessment methods it is important to define the load conditions and their frequency.

8.12.3.1 *Reservoir level loading*

The reservoir levels should be partitioned to coincide with:

- The top of active conservation (full supply) level
- Flood pool of record level (also called historic high reservoir level).
- Levels at which there are changes in zoning. For example at 1 in 1000 Annual Exceedance Probability (AEP) flood in Figure 8.70(a); and at 1 in 50 AEP flood in Figure 8.70(b).
- Embankment crest level.
- Geological features which occur above a particular level in the foundation (e.g. a highly permeable gravel layer).
- Topographic features such as major changes in foundation profile if these are above the pool of record.
- Levels at which there has been recorded anomalous seepage or piezometric readings.

For flood control storages it will probably be necessary to partition at for example 1 in 100 AEP and 1 in 1000 AEP flood levels even if these do not coincide with design features. For flood control dams which have not yet experienced a flood reaching the reservoir spillway crest level the maximum level reached historically and the spillway crest levels may be used as partition boundaries.

Table 8.25 shows an example of reservoir level partitioning. For internal erosion and piping it is the annual probability that the level in the partition is exceeded which is important regardless of how long for, not the proportion of time the reservoir is

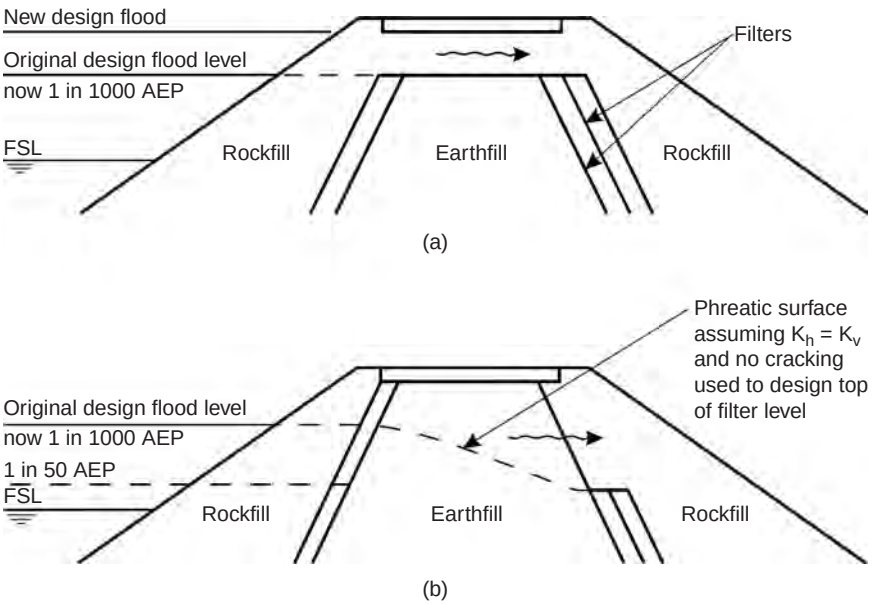


Figure 8.70 Examples of embankment crest details which may result in relatively high likelihood of internal erosion (Fell *et al.*, 2008).

Table 8.25 Example of reservoir level partitions.

Reservoir level (m)	*Significance*	*Annual exceedance probability of the reservoir level*	*Reservoir level partition range (m)*	*Probability/annum the reservoir is in this range*
RL100	Minimum pool level of record (or "Minimum pool of record")	0.9	<RL100	0.1
RL150	Top of active conservation level (or "Full pool")	0.5	RL100–RL150	0.4
			RL150–RL158	0.48
RL158	Pool of record level	0.02	(or "Historic high reservoir level")	
RL165	Embankment crest level (or "Dam crest flood")	0.0001	RL158–RL165	0.0199
			>RL165	0.0001
			Total	1.0

above the level. This is because internal erosion and piping by concentrated leak often develops quite quickly and may go from initiation to breach in hours or days.

Some risk analysts will use system response curves and a continuous AEP curve rather than use a step function approach as is implied in Table 8.25, but in any case the data is required.

Table 8.26 Example of earthquake load partitions.

Earthquake Peak Ground Acceleration	*Representative Earthquake Magnitude*	*Annual Exceedance Probability of the Earthquake Loading*	*Probability/annum the Loading is in this Stage*
			0.99
<0.10 g	6.0	0.01	
			0.009
0.20 g	6.0	0.001	
			0.0009
0.30 g	7.0	0.0001	
			0.0001
		Total	1.0

8.12.3.2 *Earthquake loading*

Earthquake loading may lead to cracking of the dam in which internal erosion may initiate. Methods for assessing the likelihood of cracking due to earthquake loads are given in Section 8.3.2.10. The important factors apart from the design of the dam, and the foundation conditions are the peak ground bedrock acceleration and earthquake magnitude.

Table 8.26 gives an example of partitioning of the earthquake loading. The partitions should be selected to represent loads which are:

- Very unlikely to cause transverse cracking in the dam.
- May cause minor transverse cracking.
- Very likely to cause significant transverse cracking.
- Likely to cause liquefaction if liquefiable zones are present in the dam or its foundation.

It will also be necessary to consider the reservoir level at the time of the earthquake. This is done by developing the plot of reservoir level versus the proportion of time the reservoir level is exceeded. This is modeled as the first node in the event tree before the earthquake loading.

In some situations the reservoir may rise to higher levels before repairs to cracks caused by seismic loading can be affected, or cracking may be undetected.

8.12.4 Potential Failure Modes Analysis (PFMA)

A potential failure mode is a sequence of events starting from an initiating mechanism, such as a defect, flaw or seepage path in the dam or its foundation, and which may lead to an uncontrolled release of the reservoir. After compiling the best available information and considering the geotechnical model of the dam or dike and foundation the dam safety assessment team should go through a discussion of all potential failure modes and develop a thorough understanding of the sequence of events and the potential location of seepage and erosion paths through the embankment and foundation. The sequence of events and seepage pathways should be documented by annotating

cross sections and longitudinal sections of the embankment and its foundation to help visualize the failure path. The development of the failure modes should consider the following:

- Potential initiating mechanisms for each of the failure locations,
- Zoning of the embankment, including the configuration and properties of internal filter and drainage measures,
- Foundation geology and stratigraphy and properties and
- Filtered and unfiltered exit points of seepage.

The possibility of overlooking potentially important failure modes is reduced by considering the particular details if the dam and it's appurtenant structures, such as details of walls retaining the embankment, conduits through the embankment, by assembling construction photographs and reports and inspecting the dam as part of the failure modes assessment. It is also reduced by having the failure modes assessment done by a team which includes the engineer and geologist most familiar with the dam, dam operating and surveillance staff, and facilitated by a person experienced in failure modes analysis.

Figure 8.71 gives examples of possible locations of initiation of internal erosion.

For many dams, it is more likely that features that could lead to initiation of internal erosion are in the upper part of the dam. This is because cracking due to differential settlement over large-scale irregularities in the foundation profile is more likely to be present near the crest. Continuation is also often more likely because the detailing of the dam design, or as built, might have given no or little filter protection to the core close to the top of the dam or the core may have been stopped too far below the crest. Figure 8.51 gives examples of this with increased likelihood of failure by internal erosion under flood loading near the crest of the dam because the earthfill core and filters were not taken to the crest of the dam.

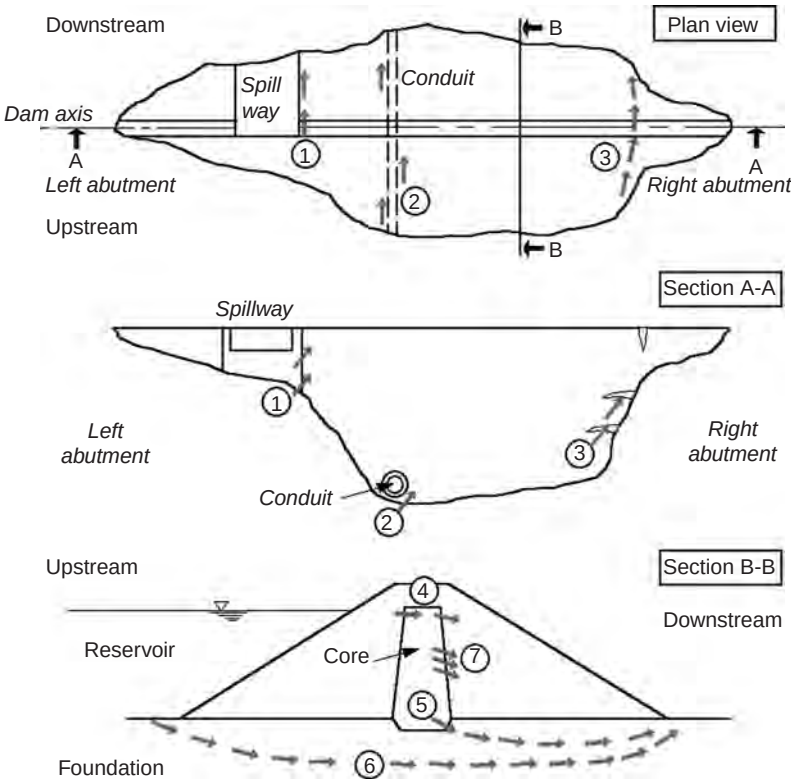
Figure 8.72 is an example where there is a significantly different probability of internal erosion above the berm than below because the upper part is essentially a homogeneous dam while the sandy gravel in zone 2 may act as a filter. This is best managed in the analysis by considering internal erosion above the berm (the upper part of the embankment) separately to below the berm (the middle and lower parts of the embankment).

8.12.5 Screening of potential failure modes

8.12.5.1 *Screening of PFM on the zoning of the dam and the properties of the core of the embankment*

(a) Zoning of the embankment

The zoning of a embankment and in particular the presence or absence of filters which satisfy modern design criteria, and are constructed to a high standard has a significant effect on the likelihood of failure by internal erosion. Figure 8.73 and Table 8.27 show typical embankment zoning and their relative likelihoods of failure related to the degree of filtering provided. These details are generalisations but it can be said that any dam with zoning in the first group has a high likelihood of failure because there is



1. Spillway wall interface
2. Adjacent to conduit
3. Crack associated with steep abutment profile
4. Desiccation on top of core
5. Embankment to foundation
6. Foundation (if the foundation is soil or erodible rock)
7. Embankment through poorly compacted layer, crack, (or by backward erosion if the core is cohesionless)

Figure 8.71 Example of possible locations of initiation of internal erosion (Fell and Fry, 2007).

no filtering, and those in the fourth group have in principle a low likelihood of failure because filters are provided. However in reality the details of the filters are critical for this group and for groups 2 and 3 and dams should not be regarded as low likelihood of internal erosion just because their general zoning is favourable.

(b) Screening of PFM on the properties of the core of the embankment

Table 8.28 shows PFM which can be screened for internal erosion in the embankment. If these criteria are met the only PFM which have to be considered are concentrated leak PFM.

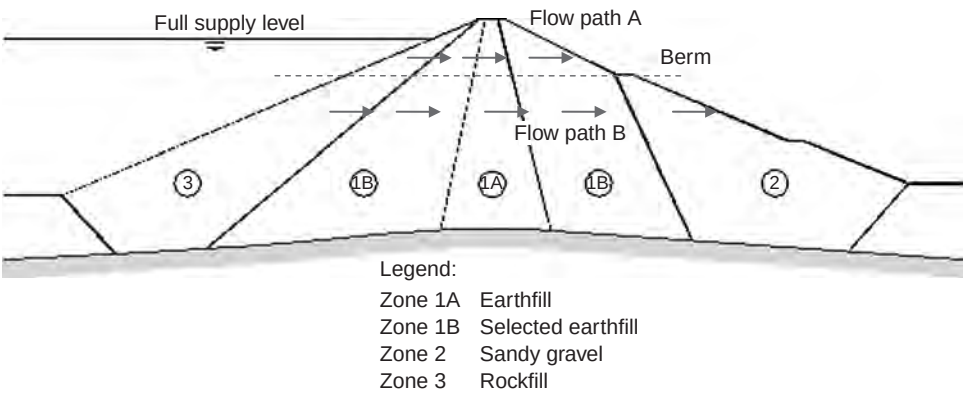


Figure 8.72 Example of an embankment with significantly different likelihood of internal erosion above and below the top of the downstream berm (Fell *et al*., 2008).

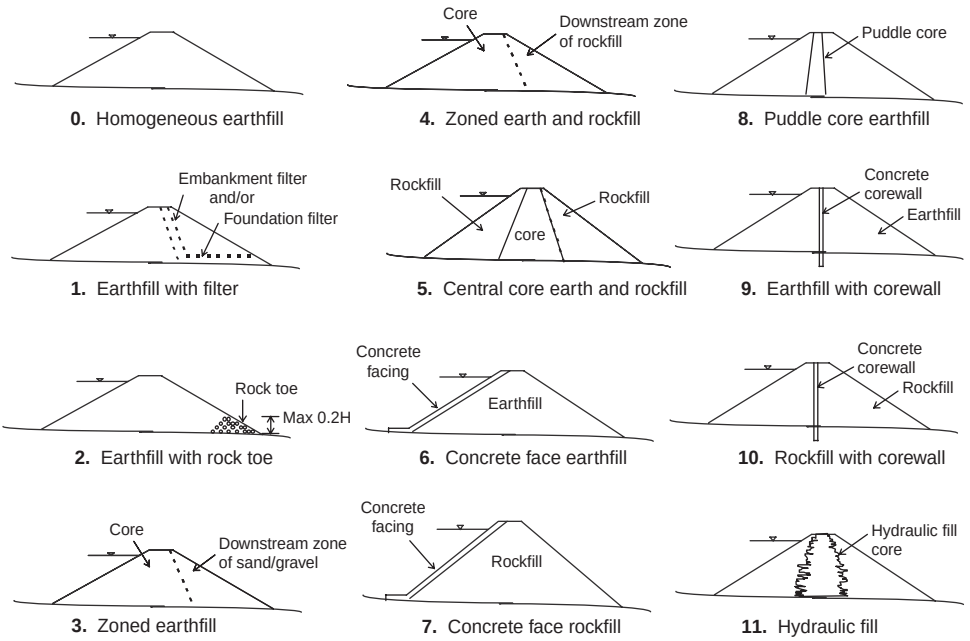


Figure 8.73 Dam zoning categories (Fell *et al*., 2008).

8.12.5.2 *Screening of PFM on foundation geology and properties*

Tables 8.29 and 8.30 show how PFM for internal erosion in the foundation, and from embankment to foundation can be screened based on the geology of the foundation and the cut-off provided for the core of the embankment.

Table 8.27 Classification of likelihood of internal erosion associated with the dam zoning.

Likelihood of Internal Erosion	*Control for internal erosion*	*Dam zoning and category number*
Large	Little or no control	Homogeneous earth fill (category 0) Earth fill with rock toe (category 2)
Moderate	Some control of internal erosion depending on detail of zoning and filter capability	Zoned earth fill (3) Zoned earth and rock fill (4) Puddle core (8) Hydraulic fill (11)
Low	Moderate control of internal erosion depending on the filter capacity and details of the core wall or face slab	Concrete face earth fill (6) Concrete face rock fill (7) Concrete core earth fill (9) Concrete core rock fill (10)
Very Low	Good control of internal erosion subject to good details of zoning and filter design	Earth fill with filters (1) Central core earth and rock fill (5)

Table 8.28 Screening of PFMA for internal erosion in the core of the embankment based on soil classification.

Initiating Mechanism	*Exclude the Failure Mode if the Following Conditions are Satisfied*
Backward erosion in the core	Exclude if the soil has a Plasticity Index $\geq$7.
Suffusion in the core	(1) Exclude if the soil has a Plasticity Index $\geq$7. OR (2) The proportion of the finer fraction of a non-plastic soil is less than 40% of the total mass of the soil.

Table 8.29 Screening of PFM for internal erosion in the foundation.

Initiating Mechanism	*Exclude the Failure Mode if the Following Conditions are Satisfied*
All modes of internal erosion of the foundation (backward erosion, suffusion, erosion in a crack)	Exclude if the soil layer beneath the dam is isolated by a cut-off trench founded in non-erodible rock
Backward erosion in a soil in the foundation	Exclude if (1) The foundation soil has a Plasticity Index $\geq$ 7. OR (2) If the soil layer with PI $\leq$ 7 layer is not continuous below the embankment (i.e. it terminates beneath the dam)
Suffusion in a soil in the foundation	Exclude if (1) The foundation soil has a Plasticity Index $\geq$ 7. OR (2) The proportion of the finer fraction of a non-plastic soil is less than 40% of the total mass of the soil. OR (3) If the soil layer with a PI $\leq$ 7 is not continuous below the embankment (i.e. it terminates beneath the dam)
Erosion in a crack in soil in the foundation	Exclude if the foundation soil is non plastic.

Table 8.30 Screening of PFM for internal erosion from the embankment to the foundation and contact erosion.

Initiating Mechanism	*Exclude the Failure Path if the Following Conditions are Satisfied*
Internal erosion of the embankment into or at a rock foundation	Exclude if (1) Rock foundation below the core is comprised of rock containing closed rock defects (< 1 mm wide) or defects open less than $3D_{95}$ of the fine limit of the core. OR (2) Rock foundation below the core has been adequately treated (e.g. shotcrete, slush grouting mortar treatment)
Internal erosion of the embankment into or at a soil foundation	Exclude if (1) Soil foundation below the core is comprised of fine-grained soils with greater than 12% fines (fraction finer than No 200 sieve, 0.075 mm), and the soil does not contain macrostructure such as root holes, relic joints or solution features. OR (2) Soil foundation below the core is comprised of sands (SP or SW) which are filter-compatible with the embankment materials (i.e. satisfy the No Erosion criteria).

8.12.5.3 *Screening of PFM on details of the embankment foundation geometry, compaction of the core, and conduits and retaining walls*

The likelihood of an embankment experiencing initiation of concentrated leak erosion will be low provided that the conditions described in Table 8.31 are ALL satisfied, AND the soil in the embankment is Category 3 or 4, moderately erodible or erosion resistant, as determined from Table 8.32. These are based on quantitative assessments in Fell *et al.* (2008).

Table 8.33 shows how PFM can be screened for failure modes related to conduits and walls retaining the dam core.

This is not to say these PFM cannot occur and for high consequence of failure dams they should not be excluded without more detailed analysis.

8.12.6 Estimation of likelihoods of failure for the Potential Failure Modes applicable to the dam

8.12.6.1 *Some general principles*

As discussed in Section 8.2.1 for a dam to fail erosion has to initiate, continue (filters are not effective), progress, intervention fails, and dam breach occurs.

The likelihood this may occur will be dependent on the reservoir level, or seismic loading.

The likelihood of failure may be assessed by engineering judgement, qualitative or quantitative Risk analysis methods taking account the frequency of the applied reservoir and seismic loads.

Whichever approach is used it is useful to consider the internal erosion process as a sequence of events as shown in flow charts in Figures 8.74 to 8.77.

Table 8.31 Screening of PFM for conditions in which a crack or concentrated leak in the embankment is unlikely.

Initiating Mechanism	*Exclude the Failure Path if the Following Conditions are Satisfied*
Transverse cracking due to cross valley differential settlement	Exclude if the embankment abutments are flatter than 15 degrees, and the embankment height is uniform across the valley
Transverse cracking due to cross valley arching	Exclude if the width of valley to dam height ratio ($W_v/H > 2$)
Transverse cracking due to differential settlements in the foundation beneath the core	Exclude if there is no compressible soil in the foundation below the core
Cracking in the crest due to desiccation by drying	Exclude if the reservoir level is below the likely depth of desiccation cracking under all conditions including during extreme floods
Cracking due to earthquake	Exclude if the MDE is below a peak ground acceleration of 0.2 g, AND The embankment abutments are flatter than 15 degrees, and the height is uniform across the valley; AND The soils in the embankment and its foundation are not susceptible to liquefaction
Transverse cracking at the foundation contact due to small scale irregularities in the foundation profile under the core	Exclude if the persistence of the irregularity across the width of the core is less than 50% of the core base width
Poorly compacted or high permeability layer in the embankment	Exclude if all soils are very well compacted with lift thicknesses less than 200 mm, with good documentation and records. This means: (1) For plastic soils (Plasticity Index > 7), ≥98% standard dry density ratio, moisture content 2% dry to 1% wet of OWC. (2) For non plastic soils and soils with PI ≤ 7, >75% relative density
Poorly compacted or high permeability layer on the core-foundation contact	Exclude if (1) Contact soils are well compacted on a regular foundation surface with good documentation and records. OR (2) Uniform or regular rock surface or surface treated with shotcrete orconcrete to correct slope irregularities, and soils well compacted (contact soil compacted using special compaction methods (e.g. rubbertires, use more plastic material, compaction wet of OWC). OR (3) Uniform well compacted soil foundation, with good mixing, bonding and compaction of contact fill. OR par (4) Compacted soil foundation
Poorly compacted or high permeability layers in the crest due to freezing	Exclude if The climate is such that temperatures do not fall below freezing point except possibly overnight or for a day or two. OR If the reservoir stage being considered is below the likely depth of freezing

If quantitative risk analysis methods are being used this can be represented in an event tree. Figure 8.78 gives an example. The first node of the tree is the annual probability the reservoir will be in this range of levels. Separate event trees are required for each of the reservoir level ranges.

Table 8.32 Erosion resistance of soils related to classification and dispersivity.

Erosion Soil Group	*Soil Classification*
1. Extremely erodible	All dispersive soils; Sherard pinhole classes D1 and D2 or Emerson Crumb Class 1 and 2 AND SM with <30% fines
2. Highly erodible	SM with >30% fines, SC with <30% fines, ML, SC with >30% fines, CL-ML
3. Moderately Erodible	CL, CL-CH, MH, CH with Liquid Limit <65%
4. Erosion resistant	CH with Liquid Limit >65%

Table 8.33 Screening of PFM for internal erosion around and into conduits through the embankment or adjacent a wall supporting the embankment core for which internal erosion is unlikely.

Poorly compacted or high permeability zone around a conduit through the embankment	Exclude if (1) There is no conduit passing through the embankment, OR (2) The conduit is totally embedded in a trench excavated in non-erodible rock, backfilled to the surface with concrete
Erosion into a (non-pressurized) conduit	Exclude if (1) There is no conduit passing through the embankment, OR (2) Careful internal inspection of conduit showing no evidence of open joints or cracks
Poorly compacted zone associated with a spillway or abutment wall	Exclude if there is no spillway or abutment wall in contact with the embankment
Crack/gap adjacent to a spillway or abutment wall	Exclude if there is no spillway or abutment wall in contact with the embankment

It will be noted that the event tree branches at the continuing erosion branch to allow dams for which the filter/transition system does not satisfy modern design criteria and there is some likelihood the filters will be in no, some, excessive and continuing erosion. The sum of these probabilities = 1.0.

The probabilities of not detecting and intervening, and breach are conditional on which continuing erosion branch is being followed and therefore are modelled separately.

The annual probability of failure for this failure mode is the product of the assigned conditional probability values together with the appropriate annual probability for the water level or earthquake event summed for all of the "yes" branches for all of the event trees.

Separate event trees are required for all PFM and the overall annual probability of failure from internal erosion is the sum of all the annual probabilities for the PFM.

8.12.6.2 *Summary of how to estimate conditional probabilities within the event tree*

Table 8.34 summarizes suggestions for how probabilities of initiation, continuation, progression, not detect and intervene, and breach may be estimated.

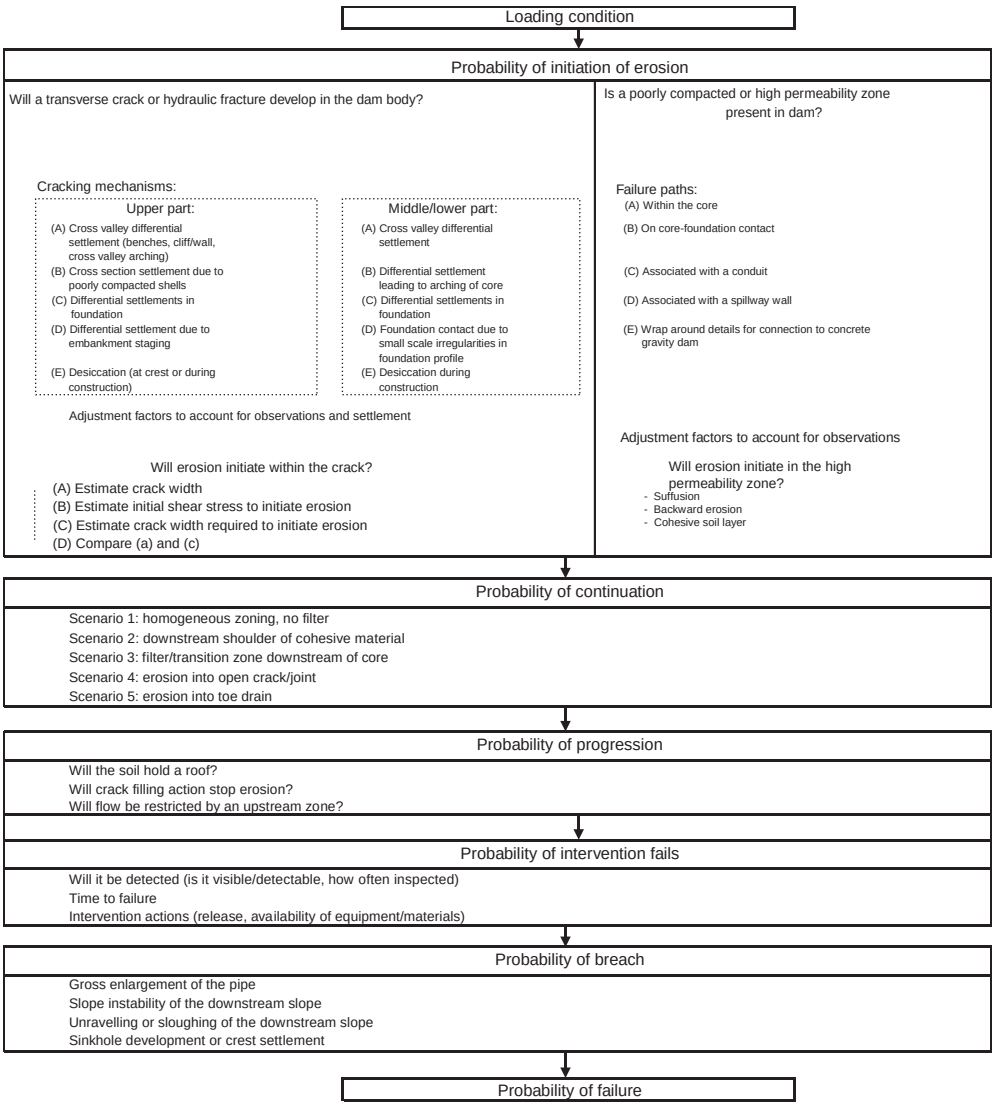

Loading condition
Probability of initiation of erosion
Will a transverse crack or hydraulic fracture develop in the dam body?
Cracking mechanisms:
Upper part:
(A) Cross valley differential settlement (benches, cliff/wall, cross valley arching)
(B) Cross section settlement due to poorly compacted shells
(C) Differential settlements in foundation
(D) Differential settlement due to embankment staging
(E) Desiccation (at crest or during construction)
Middle/lower part:
(A) Cross valley differential settlement
(B) Differential settlement leading to arching of core
(C) Differential settlements in foundation
(D) Foundation contact due to small scale irregularities in foundation profile
(E) Desiccation during construction
Adjustment factors to account for observations and settlement
Will erosion initiate within the crack?
(A) Estimate crack width
(B) Estimate initial shear stress to initiate erosion
(C) Estimate crack width required to initiate erosion
(D) Compare (a) and (c)
Is a poorly compacted or high permeability zone present in dam?
Failure paths:
(A) Within the core
(B) On core-foundation contact
(C) Associated with a conduit
(D) Associated with a spillway wall
(E) Wrap around details for connection to concrete gravity dam
Adjustment factors to account for observations
Will erosion initiate in the high permeability zone?
- Suffusion
- Backward erosion
- Cohesive soil layer
Probability of continuation
Scenario 1: homogeneous zoning, no filter
Scenario 2: downstream shoulder of cohesive material
Scenario 3: filter/transition zone downstream of core
Scenario 4: erosion into open crack/joint
Scenario 5: erosion into toe drain
Probability of progression
Will the soil hold a roof?
Will crack filling action stop erosion?
Will flow be restricted by an upstream zone?
Probability of intervention fails
Will it be detected (is it visible/detectable, how often inspected)
Time to failure
Intervention actions (release, availability of equipment/materials)
Probability of breach
Gross enlargement of the pipe
Slope instability of the downstream slope
Unravelling or sloughing of the downstream slope
Sinkhole development or crest settlement
Probability of failure

Figure 8.74 Flowchart for internal erosion in the embankment.

Appendix A has details of how these may estimated. These are taken from Fell *et al*. (2008) and should be used as described in the Appendix. They are semi empirical and should be regarded as aids to judgement.

8.12.6.3 *Ways in which the safety of the dam against internal erosion and piping can be considered*

There are a number of ways the safety of the dam against internal erosion and piping can be considered.

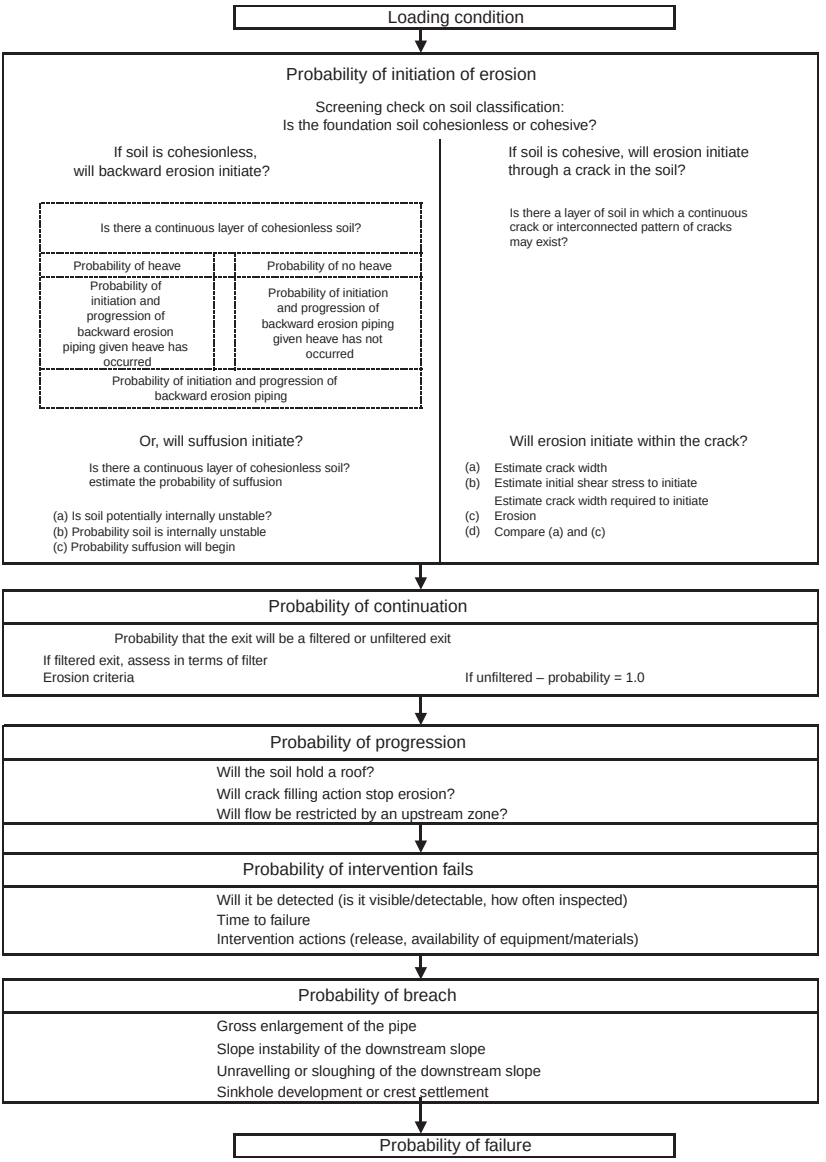

Loading condition
Probability of initiation of erosion
Screening check on soil classification:
Is the foundation soil cohesionless or cohesive?
If soil is cohesionless,
will backward erosion initiate?
If soil is cohesive, will erosion initiate
through a crack in the soil?
Is there a continuous layer of cohesionless soil?
Is there a layer of soil in which a continuous
crack or interconnected pattern of cracks
may exist?
Probability of heave
Probability of no heave
Probability of
initiation and
progression of
backward erosion
piping given heave has
occurred
Probability of initiation
and progression of
backward erosion piping
given heave has not
occurred
Probability of initiation and progression of
backward erosion piping
Or, will suffusion initiate?
Will erosion initiate within the crack?
Is there a continuous layer of cohesionless soil?
estimate the probability of suffusion
(a) Estimate crack width
(b) Estimate initial shear stress to initiate
Estimate crack width required to initiate
(c) Erosion
(d) Compare (a) and (c)
(a) Is soil potentially internally unstable?
(b) Probability soil is internally unstable
(c) Probability suffusion will begin
Probability of continuation
Probability that the exit will be a filtered or unfiltered exit
If filtered exit, assess in terms of filter
Erosion criteria
If unfiltered – probability = 1.0
Probability of progression
Will the soil hold a roof?
Will crack filling action stop erosion?
Will flow be restricted by an upstream zone?
Probability of intervention fails
Will it be detected (is it visible/detectable, how often inspected)
Time to failure
Intervention actions (release, availability of equipment/materials)
Probability of breach
Gross enlargement of the pipe
Slope instability of the downstream slope
Unravelling or sloughing of the downstream slope
Sinkhole development or crest settlement
Probability of failure

Figure 8.75 Flowchart for internal erosion in a soil foundation.

(a) Engineering judgement approach

This would involve using the following information:

- The embankment zoning and material classification, and particle size distributions of the core, filters and other zones in the dam.
- The dispersion properties of the core.

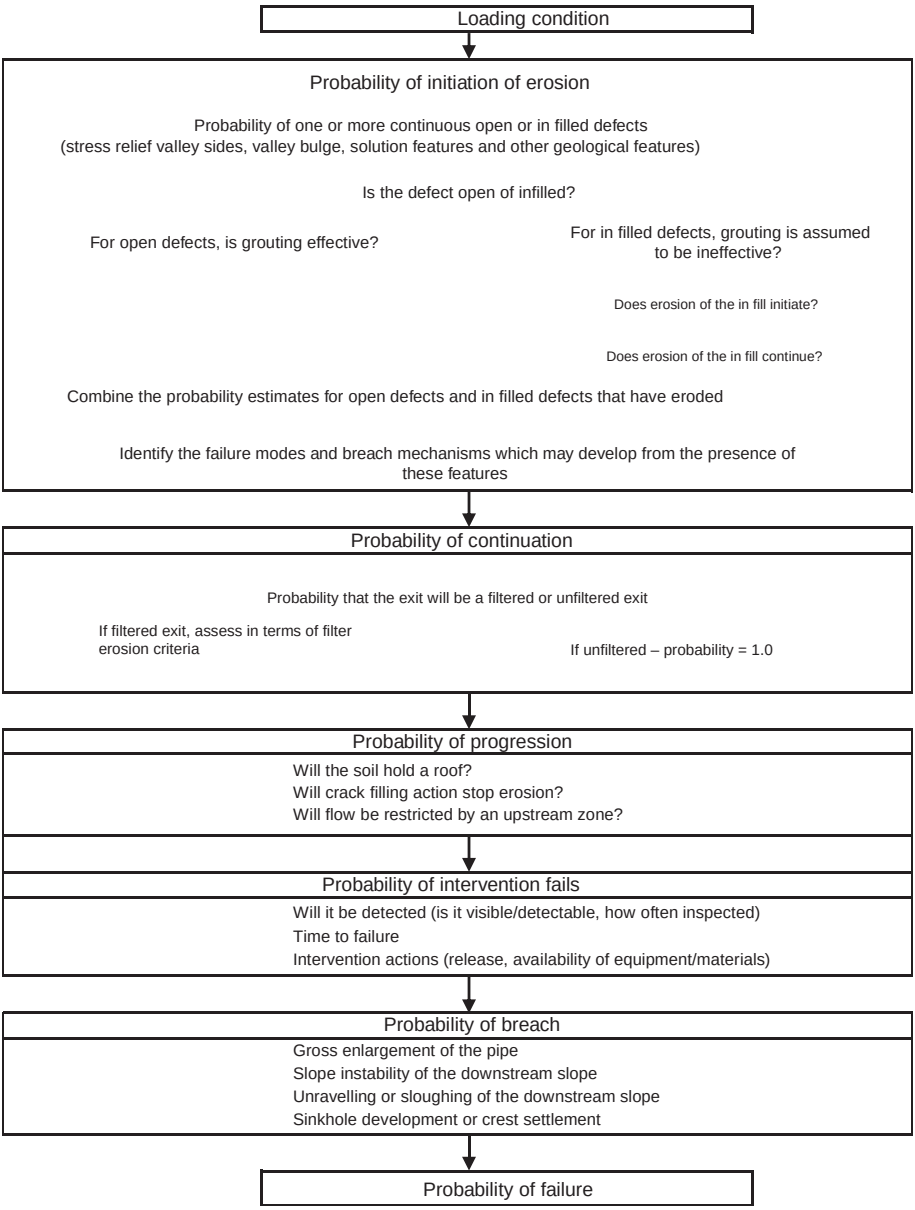

Loading condition
Probability of initiation of erosion
Probability of one or more continuous open or in filled defects
(stress relief valley sides, valley bulge, solution features and other geological features)
Is the defect open of infilled?
For open defects, is grouting effective?
For in filled defects, grouting is assumed to be ineffective?
Does erosion of the in fill initiate?
Does erosion of the in fill continue?
Combine the probability estimates for open defects and in filled defects that have eroded
Identify the failure modes and breach mechanisms which may develop from the presence of these features
Probability of continuation
Probability that the exit will be a filtered or unfiltered exit
If filtered exit, assess in terms of filter erosion criteria
If unfiltered – probability = 1.0
Probability of progression
Will the soil hold a roof?
Will crack filling action stop erosion?
Will flow be restricted by an upstream zone?
Probability of intervention fails
Will it be detected (is it visible/detectable, how often inspected)
Time to failure
Intervention actions (release, availability of equipment/materials)
Probability of breach
Gross enlargement of the pipe
Slope instability of the downstream slope
Unravelling or sloughing of the downstream slope
Sinkhole development or crest settlement
Probability of failure

Figure 8.76 Flowchart for internal erosion through a rock foundation.

- The foundation geology and geotechnical properties such as classification and particle size distributions.
- Details of conduits which penetrate through, and walls which retain the core of the dam.

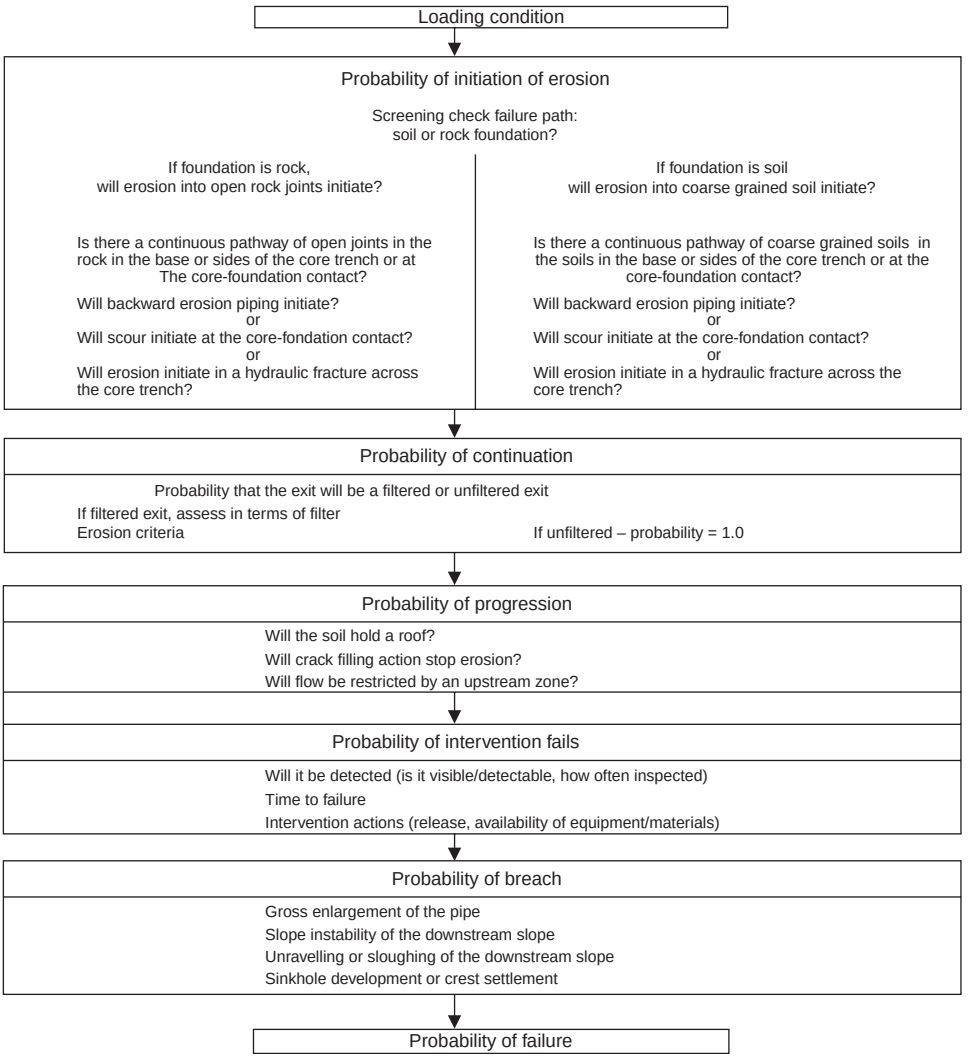

Loading condition
Probability of initiation of erosion
Screening check failure path:
soil or rock foundation?
If foundation is rock,
will erosion into open rock joints initiate?
Is there a continuous pathway of open joints in the rock in the base or sides of the core trench or at The core-foundation contact?
Will backward erosion piping initiate?
or
Will scour initiate at the core-fondation contact?
or
Will erosion initiate in a hydraulic fracture across the core trench?
If foundation is soil
will erosion into coarse grained soil initiate?
Is there a continuous pathway of coarse grained soils in the soils in the base or sides of the core trench or at the core-foundation contact?
Will backward erosion piping initiate?
or
Will scour initiate at the core-fondation contact?
or
Will erosion initiate in a hydraulic fracture across the core trench?
Probability of continuation
Probability that the exit will be a filtered or unfiltered exit
If filtered exit, assess in terms of filter
Erosion criteria
If unfiltered – probability = 1.0
Probability of progression
Will the soil hold a roof?
Will crack filling action stop erosion?
Will flow be restricted by an upstream zone?
Probability of intervention fails
Will it be detected (is it visible/detectable, how often inspected)
Time to failure
Intervention actions (release, availability of equipment/materials)
Probability of breach
Gross enlargement of the pipe
Slope instability of the downstream slope
Unravelling or sloughing of the downstream slope
Sinkhole development or crest settlement
Probability of failure

Figure 8.77 Flowchart for internal erosion of the embankment into or at the foundation.

- The annual probability that the reservoir level will reach critical levels, such at the top of filters in Figure 8.70(a) or critical gradients for backward erosion piping or suffusion in the foundation if those are PFM.
- The likelihood that the filters in the embankment will be no erosion, some erosion, excessive erosion or continuing erosion using the methods given in Section 9.3.4.
- The consequence category of the dam.

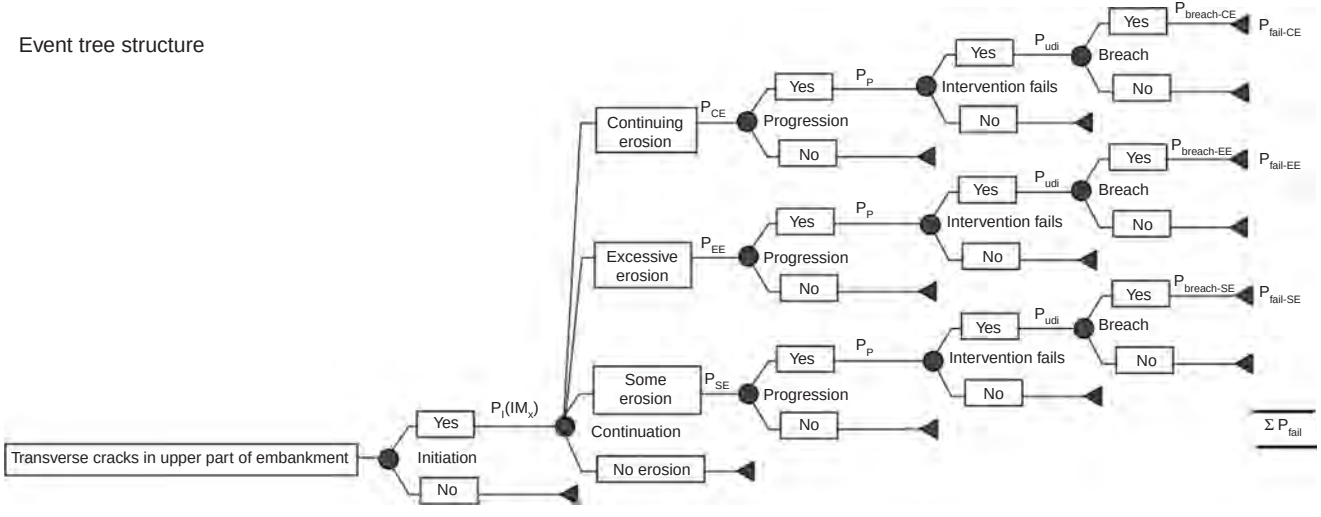


Figure 8.78 Event tree structure for internal erosion in the upper part of an embankment resulting from concentrated leak erosion.

Table 8.34 Summary of how annual probabilities of initiation, continuation, progression, not detect and intervene, and breach should be estimated.

Stage in erosion process	*Section(s) where method for estimating probability is described*
Initiation	
Concentrated Leak[(1)]	8.3.2 Situations where cracking and low stress zones may be present in an embankment or the foundation
	8.3.3 Estimation of crack width and depth of cracking
	8.3.4 The Mechanics of erosion in concentrated leaks
	8.3.5 Comparison of the hydraulic shear stress in the crack (τ) to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c)
Backward Erosion	8.4.3 Methods for predicting whether backward erosion piping will initiate and progress
	8.4.4 Methods for prediction of initiation and progression of global backward erosion
Suffusion	8.5.2 Methods of identifying soils which are internally unstable and potentially subject to suffusion
	8.5.4 Assessment of the seepage gradient which will cause suffusion
Contact Erosion	8.6.2 Methods for predicting initiation and progression of contact erosion
Continuation[(1)]	
	9.3.4 Assessment of the likelihood of continuation where a Filter/ transition zone do not satisfy no-erosion filter criteria
	9.3.5 Assessment of the likelihood of continuation for internal erosion into an open defect, joint or crack in the foundation, in a wall or in a conduit
Progression[(1)]	
Overall Approach	8.8.2 Overall approach for assessing progression for concentrated leak erosion
Hold a roof	8.8.3 Assessing whether the soil will hold a roof to a developing pipe
Crack filling	8.8.4 Assessing whether crack filling action will occur
Upstream flow limitation	8.8.5 Assessing whether upstream flow limitation will occur
Detection and Intervention Fails[(1)]	
	8.9.3 The likelihood of detection and intervention
Breach[(1)]	
	8.10 Initiation of breach

Note: [(1)]Appendix A has detailed information on how to estimate these probabilities

This information would be considered by the dam engineers and other stakeholders such as the owner, the organisation which uses the water from the dam, and the dam safety regulator to make a judgement whether the dam was tolerably safe.

(b) Enhanced engineering judgement approach

This would involve the information listed above for "Engineering judgement approach" and some or all of the following information:

- Use of the methods described in Sections 8.3.2 and 8.3.3 and Appendix A to estimate the probability of cracking, and the likely depth of cracking due to the PFM identified for the dam, including cross valley differential settlement.

- Use of the methods described in Section 8.3.3 and Appendix A to estimate the probability of poorly compacted zones in the core generally, surrounding conduits penetrating through the core, and adjacent walls retaining the core.
- Use of the data in Section 8.3.4.3 to determine the likely erosion properties of the core.
- Use of the methods described in Section 8.9.3 and Appendix A to assess the likelihood of successful detection and intervention.

This additional information gives a greater insight to the likelihood that cracking may be present, and how deep below the crest of the dam it may persist. This can then be related to the AEP that the reservoir will reach these cracks. It would be considered along with the other data as described above.

(c) Risk enhanced engineering judgement approach

This would involve the information listed above for "Enhanced engineering judgement approach" and some or all of the following information:

- Use of the methods in Section 8.3.5 and Appendix A to estimate the likelihood of initiation of erosion in cracks.
- Combine this with the in information on the depth and width of cracking, and the AEP reservoir levels will be exceeded to estimate the annual probability of initiation of internal erosion for each of the PFM.
- Combine this with the likelihoods of the filters being no erosion, some erosion, excessive erosion and continuing erosion to assess the annual probabilities of initiation and continuation of erosion in each category.
- Use the methods described in Section 8.8.2 to 8.8.5 and Appendix A to estimate the probability of progression.
- Use the methods described in Section 8.10 and Appendix A to estimate the probability of breach. This and the probability of detection and intervention will be dependent on which branch of continuation is involved, with many dams able to cope with the small leakage flows for some and even excessive erosion conditions.

The advantage of this approach over the previous two is that there is a degree of quantification of the annual probabilities of failure which aids in the risk assessment process.

(d) Quantitative risk assessment approach

This would involve the information listed above for Risk enhanced engineering approach, but the annual probabilities of failure for the PFM would be formally estimated and summed. The consequences of the dam failing would be quantified as expected lives lost, and financial and economic losses. The methods for doing this are discussed in Section 8.11.6.4 and detailed in Fell *et al.* (2008).

The annual probability of failure and expected life loss would be used to assess the adequacy of the dam using societal and individual risk criteria such as those in ANCOLD (2003a) which are reproduced in Appendix A.

8.12.6.4 *Quantitative risk analysis methods for internal erosion and piping*

Methods based on historic databases of failure and incidents

UNSW Method. The UNSW method is described in Foster *et al.* (2000(a), (b)). The database of dam incidents was compiled primarily from the three ICOLD studies (ICOLD 1974, 1983 and 1995) and supplemented with additional incident cases from the other existing compilations, from the literature and dam engineering organisations.

The method allows estimation of the annual probability of a dam failing within the embankment, in the foundation and from embankment to foundation based on the dam zoning, the material in the core, the filters, foundation geology and cut-off details.

The method allow a broad categorization of the dam into those less and more likely to fail. However the authors have found that it does not enable important details of the dam to be allowed for and it cannot model the frequency of reservoir loading which is an important factor in estimating the likelihood of failure. In view of this we prefer to use event tree methods such as the "piping toolbox" described below.

Interim Guide to Quantitative Risk Assessment for UK Reservoirs. The Interim Guide to Quantitative Risk Assessment for Reservoirs in the United Kingdom (Brown and Gosden, 2004, 2008), is a quantitative method, based on analysis of the historic performance of the large population of similar dams in the United Kingdom, for which records of incidents of poor performance have been kept for many years.

The method is only applicable to the dams in the UK and suffers from the same limitations as the UNSW method.

Event tree methods. Event tree methods have been used by a number of organisations including USBR, USACE, and Australian Dam organisations since the late 1990s. The methods have been refined with time and are still being refined as the understanding of the mechanics of internal erosion is improved.

These methods mostly rely on "expert opinion elicitation" to estimate conditional probabilities within the event trees. They are successful provided those doing the analysis are familiar with the mechanics of internal erosion and quantitative risk analysis. Because of this their use has been largely restricted to USBR, USACE, and Australian and some USA and UK Consultants.

The "Piping toolbox" or "Risk analysis for Dam Safety, A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal erosion and Piping" is an event tree system for estimating the annual probability of breach of embankment dams by internal erosion and piping in the embankment, the foundation and from the embankment into the foundation.

It was developed as a unified approach for carrying out risk analyses by the United States Bureau of Reclamation (USBR), United States Army Corps of Engineers (USACE), Consulting Engineers URS and the University of New South Wales.

The document is in two parts, a Guidance Document, and a Supporting Information Document. It is available as a University of New South Wales UNICIV Report in electronic form. The full title of this document is:

"Fell, R., Foster, M., Davidson, R., Cyganiewicz, J., Sills, G. and Vroman, N. (2008) A Unified Method for Estimating Probabilities of Failure of Embankment Dams

by Internal Erosion and Piping. UNICIV Report R 446, The School of Civil and Environmental Engineering, the University of New South Wales, Sydney, Australia 2052. ISBN:85841 413 9"

The method was used within the USACE for a number of dams, and is in continuing use in Australia. The USBR and USACE have now preferred to use it only as a reference document. The reasons for this are: (C. Redlinger, personal communication)

"Internal erosion is a very complex phenomenon that is best evaluated considering multiple approaches with recognition that each method has strong points and shortcomings that need consideration before arriving at a decision. The methods outlined in the best practices document allows for inputs from multiple approaches.

The use of the current toolbox as well as previous tools that provide scripted approaches to solutions, albeit with good intentions related to consistent solutions, has stifled teams from reviewing and referencing past incidents and failures that contain lessons that are applicable to the dam under study. In addition, the thought, discussion and "learning" that should accompany evaluating these types of failure modes were not occurring.

Scripted approaches, once turned into a spreadsheet, have also unintentionally resulted in teams focusing more on the risk estimates and less on the identification of strengths and weakness of a dam and/or foundation. In many reviews, the risk estimators have stated that their estimate is what came out of the toolbox or spreadsheet and are unable to build the case for the risk estimates and planned actions.

At the time the development of a "unified toolbox" was initiated, it was envisioned as a means to obtain a consistent approach and results while using a large pool of risk estimators estimating internal erosion risks across a very diverse and large inventory of embankment dams within our agency as well as others. Through the review of analyses done using the toolbox by highly experienced dam engineers familiar with risk analysis, our agency (the USACE) has identified significant issues associated with using the current toolbox some of which are identified above.

Changes to the organization of the agency, (the USACE) including the recent formation of the Risk Management Center, have allowed the agency to implement the approaches outlined in the USACE best practices document more directly in arriving at risk informed decisions."

The authors experience in Australia and USA is that without guidance methods such as the Piping Toolbox estimates of probabilities of internal erosion and piping are very dependent on the team doing the estimates. Also few dam engineers, even very experienced ones, have a complete understanding of internal erosion and piping, and even fewer any feel for the likelihood of cracking and initiation of concentrated leak erosion as a result.

The authors are of the view that the issues raised by USACE are real but can be overcome by training, and that if that is done the Toolbox gives more consistent and reasonable results than expert judgement alone. The authors do not use a spreadsheet preferring to use the document directly as this means those doing the analyses can

see clearly the steps involved. This approach also forces the engineers "to think very carefully" about the problem at hand and not to do their assessment by rote and a "black box" that produces the final numbers.

When applying the Toolbox engineering judgement should be applied to the results to make sure they seem reasonable.

The sections in the Toolbox on backward erosion piping are based on the Schmertmann (2000) method and the authors are now not satisfied that method is soundly based. We prefer to use the Sellmeijer and co-workers methods and estimate probabilities of initiation and progression based on calculations of critical gradients, the actual gradients for each reservoir level and allow for the uncertainties in the inputs and the method.

For internal erosion in and into rock foundations the Toolbox "fall back" method gives too high probabilities of failure, at least in the hands of less experienced engineers. This is due in part to limited calibration of those parts of the toolbox during the development stage. It was expected that this part of the Toolbox would be revised after trialling but that has not eventuated. In view of this the authors use expert judgement for these PFM but do rely on the logic framework set out in the Toolbox.

We have found the sections in the Toolbox on initiation by concentrated leak erosion and suffusion, continuation, progression, detection and intervention, and breach give results we are comfortable with.

The Toolbox is designed for use in quantitative risk analysis. If it is to be used for that purpose the Development Team and their organisations strongly recommend that all risk analysts that will use this toolbox be trained in its use. The complexity of the issues and importance of the end product demands that all analysts fully understand the methodology.

The key goals of the training would be to provide an understanding of all features and components of the methodology; to outline for the analyst the supporting information and background that was used in the development of the methodology; and to guide the analyst through a detailed, real life example use of the methodology.

The authors of the Toolbox have seen a number of examples where the lack of adequate training has resulted in incorrect application of the methods.

Some perspectives on the use of the Toolbox in the United Kingdom are given in Eddleston *et al.* (2009, 2010).

It is recognized that not many will wish to us the document for quantitative risk analysis. It is however a valuable document for those managing the safety of dams for internal erosion and piping. The Tables of conditional probabilities which are partly reproduced in Append A are a useful guide to the factors which affect the likelihood of initiation, continuation, progression and breach and of the likelihood of successful detection and intervention.

An electronic copy of the document can be obtained from.

www.engineering.unsw.edu.au/civil-engineering/uniciv-reports

The USBR and USACE have prepared a Best Practices Training Manual (USBR 2012) for quantitative risk assessment of dam and levee safety risk analysis and this includes a chapter (Chapter 26) on internal erosion and piping. This presents useful information, tools, and techniques for estimating the probabilities for piping and internal erosion. This can be downloaded from the USBR website.

Chapter 9

Design, specification and construction of filters

9.1 GENERAL REQUIREMENTS FOR DESIGN AND THE FUNCTION OF FILTERS

9.1.1 Functional requirements

Filters in embankment dams and their foundations are required to perform two basic functions:

(a) Have the ability to control and stop the erosion which may have initiated by a concentrated leak, backward erosion, contact erosion or suffusion (internal instability) in the base soil.
(b) Have sufficient permeability (and, if a drain, thickness) to discharge the seepage flows without excessive build-up of head.

To achieve these functions the filter or filter zone will (ICOLD 1994):

– Not segregate during processing, handling, placing, spreading or compaction.
– Not change in gradation (by degradation or break down) during processing, handling, placing and/or compaction, or degrade with time, e.g. by freeze-thaw or wetting and drying by seepage flow.
– Be internally stable, that is, the fines particles in the filter should not erode from the filter under seepage flows.
– Not have any apparent or real cohesion, or ability to cement as a result of chemical, physical or biological action, so the filter will not allow a crack in the soil it is protecting to persist through the filter.

Filters are usually specified in terms of their particle size distribution. They are required to be sufficiently fine, relative to the particle size of the soil they are protecting (the "base soil"), to achieve function (a), while being sufficiently coarse to achieve function (b).

9.1.2 Flow conditions acting on filters

Figure 9.1 illustrates the basic flow conditions that can occur between a filter and base soil. These are:

N_1 Flow normal to the base soil – filter interface, with potentially high gradient conditions; e.g. at the downstream face of the earthfill core, the contact between

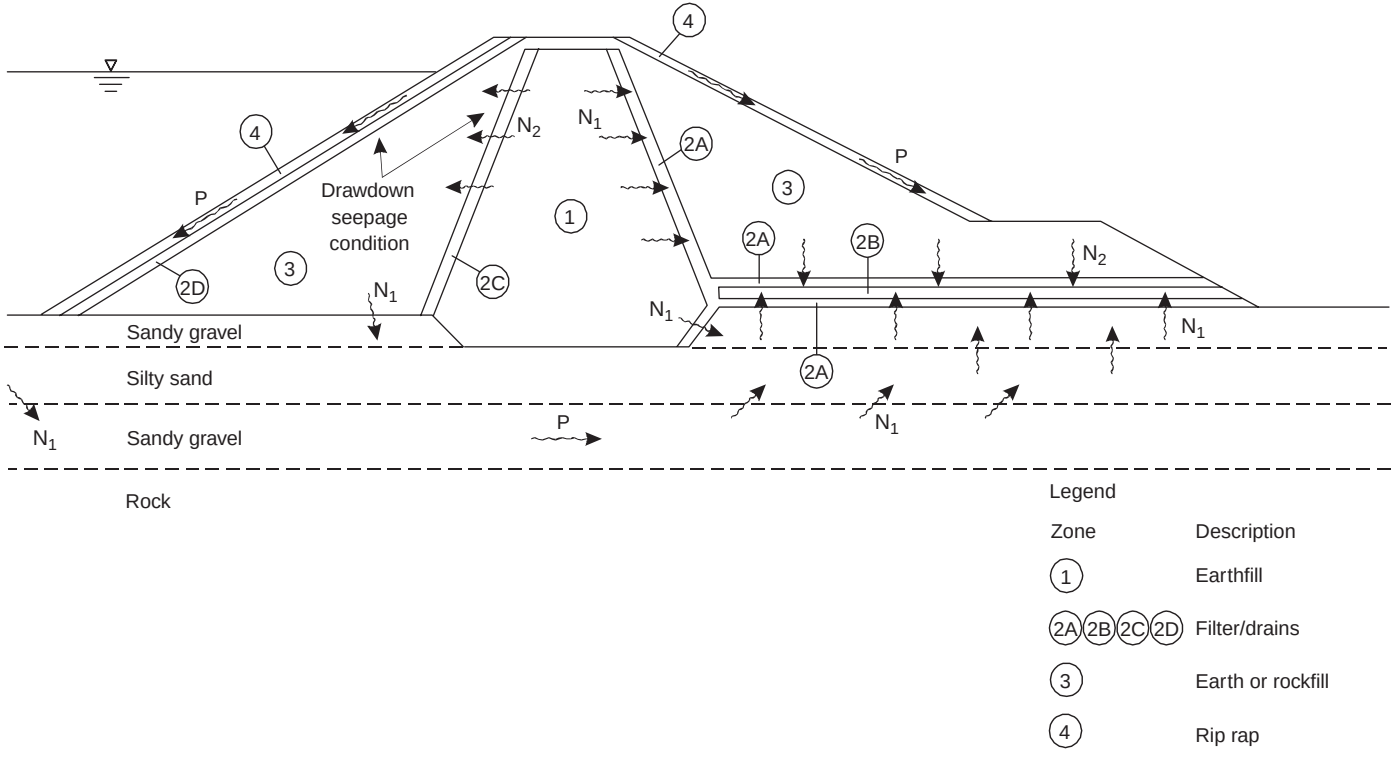


Figure 9.1 Flow conditions acting on filters. P = flow parallel to interface; N_1 = flow normal to interface, high gradient conditions; N_2 = flow normal to interface, low gradient conditions.

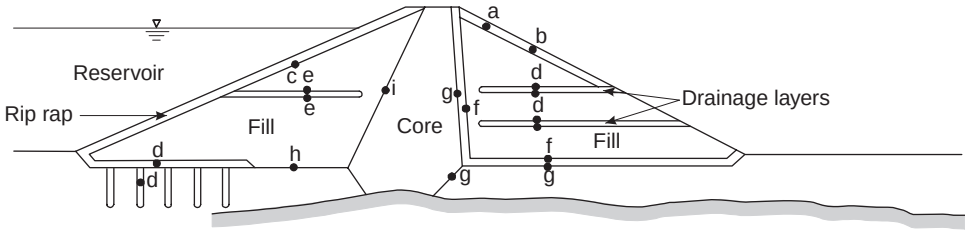


Figure 9.2 Filter functions (adapted from ICOLD, 1986).

the horizontal drain and the foundations and within the foundation where seepage is across bedding.

N_2 Flow normal to the base soil – filter interface, with low gradient conditions; e.g. at the upstream face of the earthfill core under reservoir drawdown conditions, or into the upper Zone 2A filter for the horizontal drain.

P Flow parallel to the interface, e.g. at the base of rip-rap layers, or in the foundation where seepage is along bedding within the foundation.

As will be described in Section 9.2, the erosive stresses are greatest for case N_1 and less for N_2 because for N_2 the flow is draining from the base soil under gravity, not under reservoir water head. As a result less conservative (and therefore coarser) filter may be used for cases N_2 and than for N_1.

The erosive action for case P is different. This is the contact erosion condition.

9.1.3 Critical and non critical filters

Figure 9.2 and Table 9.1 show a number of applications of filters in dams. Some filters are critical to the control of internal erosion in the dam and, if they fail, give an increased likelihood of internal erosion and piping progressing and potentially breaching the dam. These are termed critical filters, and for a new dam or remediation of an existing dam, they should be designed and constructed to meet stringent, no-erosion filter criteria. Examples are filters "g" in Figure 9.2. Some filters are non-critical, in that, if some erosion occurs, they can be repaired (e.g. beneath rip-rap, locations "a", "b" and "c" in Figure 9.2) or they will cause problems only during construction ("d" and "e", in Figure 9.2).

Most critical filters are in an N_1 flow condition (Section 9.1.2) and non-critical filters in an N_2 or P flow condition.

Filters "f" and "i" are critical to the performance of the dam, but are in N_2 flow conditions, so may be designed and constructed to lesser standards.

Filters "d" and "h" may be critical or non-critical, depending on whether or not erosion of the embankment fill into the foundation can occur. If it can, the filters are critical. As shown, "d" is a construction drainage system, so would probably be non-critical.

Much of the discussion in this chapter is about critical filters designed to control internal erosion under N_1 flow conditions.

Table 9.1 Critical and non-critical filters (adapted from ICOLD, 1986).

Filter location	*Purpose of filter*	*Type of flow or loading*	*Significance of filter*	*Access for repair*
a. Downstream slope protection	Control of erosion by rainfall	P – Occasional surface flow	Non-critical	Easy
b. Downstream surface drains	Removal of surface seepage	P – Continuous or occasional local seepage	Non-critical. Local wet areas may reappear	Easy, possible
c. Upstream slope protection	Control of erosion by wave action and by outward flow during drawdown	P – Cyclic flow during wave action. N_2 – Small flow during drawdown	Usually non-critical	Possible, but may be difficult
d. Temporary internal drainage during construction	Dissipation of excess pore pressure during construction of wet fills	N_2 – Temporary flow, limited quantity. Some migration of fines allowable if drains not blocked	Non-critical. Failure may lead to instability during construction, or delays	None
e. Upstream internal fill boundary	Prevention of unacceptable migration of fines in upstream direction	N_2 – Transient and small flows during drawdown	Non-critical. Only significant if migration is large and continuous	None
f. Downstream internal interface	Prevention of unacceptable migration of fines into filter-drains	N_2 – Flow only due to infiltration of rainfall, not from reservoir or foundation	Critical, but only if erosion is large and continuous	None
g. Downstream interface, e.g. downstream core boundary or foundation interface near core	Prevention of internal erosion of core including effects of concentrated flow in cracks, etc.	N_1 – Continuous flow from reservoir, potentially large and increasing if erosion occurs	Critical	None
h. Upstream interface between embankment and foundation	Prevention of internal erosion of core into foundation	N_1 – Continuous flow from reservoir, potentially large and increasing if erosion occurs	Critical	None
i. Upstream internal interface	Prevention of unacceptable migration of fines from core into upstream fill	N_2 – usually approaching N_1 for pumped storage dams	Critical, but only if erosion is large and continuous	None

9.1.4 Filter design notation and concepts

9.1.4.1 *Notation*

The notation used in this book is described in the following examples:

D_{15F}	Particle size of the filter material for which 15% by weight is finer.
D_{85B}	Particle size of the base material for which 85% by weight is finer.
pp% 0.075 mm	Percent finer than a particle size of 0.075 mm.
Fines content	pp% 0.075 mm.

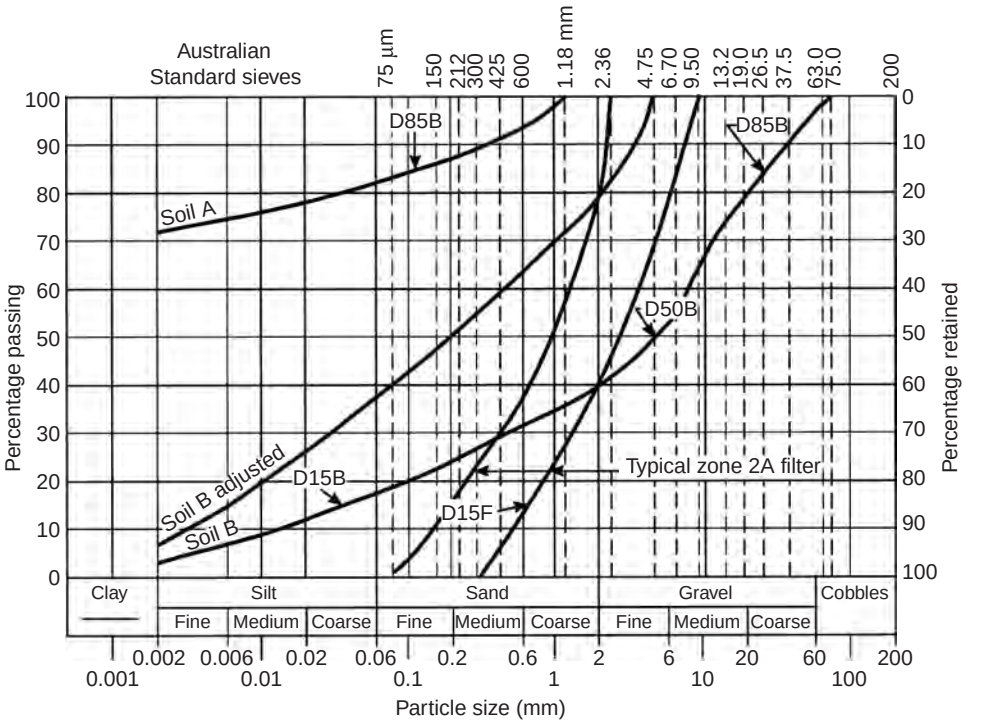


Figure 9.3 Filter design notation and adjustment of the particle size distribution for gravelly base soils.

Figure 9.3 shows particle size distributions for two base soils, A and B, and a Zone 2A filter.

For broadly graded soils, such as soil B, which have some medium and coarse gravel, the USBR (1977 and 1989a), USDA-SCS (1994), Sherard and Dunnigan (1989) and other methods use an adjusted particle size distribution. As shown in Figure 9.3, this involves taking the particle size distribution and adjusting it to what it would be if only the fraction passing the 4.75 mm sieve were used.

9.1.4.2 *Filtering concepts*

Figures 9.4(a) and (b) show the interface between a filter and base soil. The basic concept of filter design is to design the particle size distribution of the filter so that the voids in the filter are sufficiently small to prevent erosion of the base soil.

The void size in the filter is controlled by the finer particles and, for design purposes, the D_{15F} is usually used to define this void size. Sherard *et al.* (1984a) showed that for granular soils the void size between the soil particles, known as the opening size, is given by $O_E = D_{15F}/9$ (see Section 9.2.2 and 9.6.2.2). Testing by Foster (1999) confirmed this.

A further basic concept, inherent in filter design, is that the base soil will generally provide a degree of "self-filtering". Hence in Figure 9.4(a), in a well graded base soil the coarser particles in the base soil are prevented from eroding into the filter and they in turn prevent the medium sized particles in the base soil from eroding and the

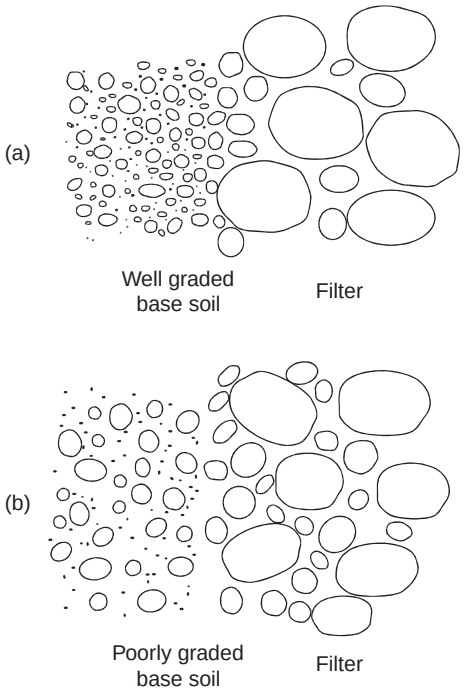


Figure 9.4 Filtering and self filtering concepts, (a) well graded base soil, (b) gap graded, or concave graded soil, deficient in medium sized particles.

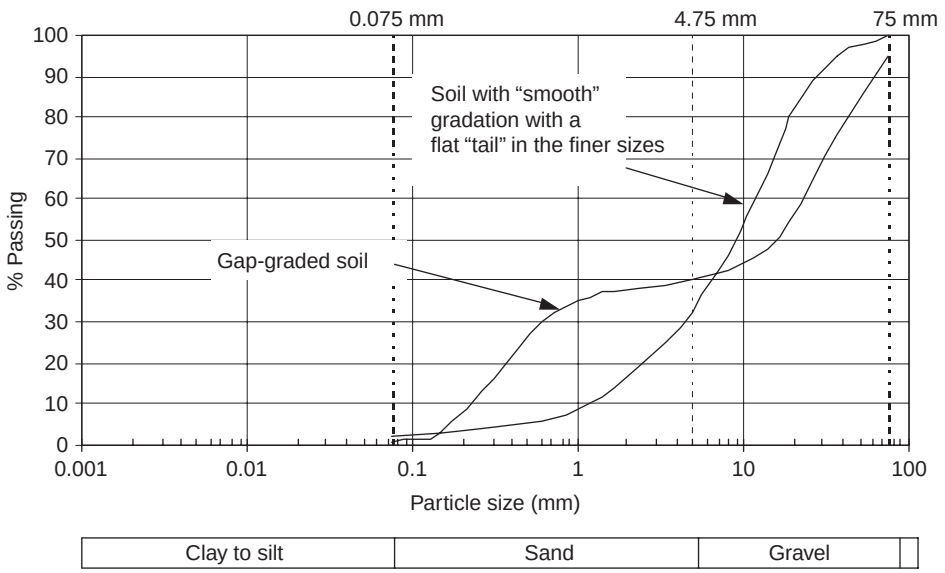


Figure 9.5 Soils gradation types which are susceptible to suffusion or internal instability.

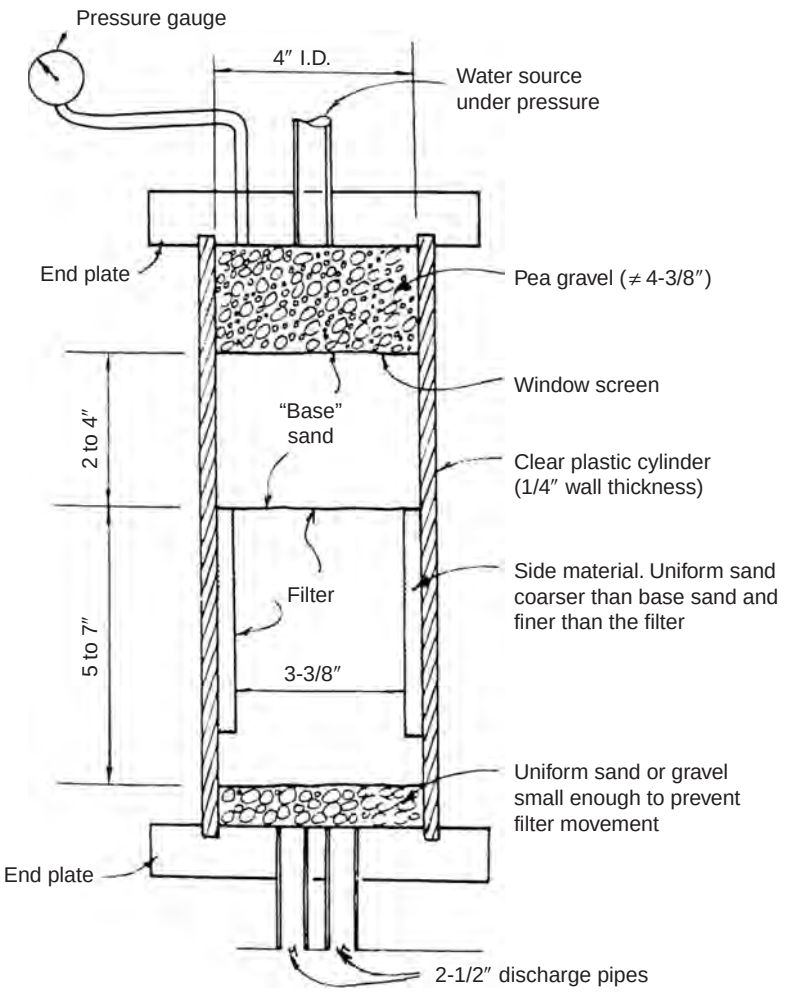


Figure 9.6 USSCS filter test apparatus details for fine to coarse sand base soils (Sherard *et al.*, 1984a, reproduced with permission of ASCE).

medium sized particles in the base soil prevent the fine particles in the base soil from eroding.

If the base soil is gap-graded or graded concave upwards (Figure 9.5), there is a deficiency of medium sized particles, as shown in Figure 9.4(b). The self filtering does not occur and the fine particles in the base soil will erode through the coarse particles, a process called suffusion or internal instability. In these situations, for the filter to be successful in controlling erosion, it must be able to control the erosion of these finer particles.

In most filter design methods the base soil particle size is characterized by the D_{85B} size, or for coarser soils, the adjusted D_{85B} as described above. Some use D_{95B} and D_{50B}

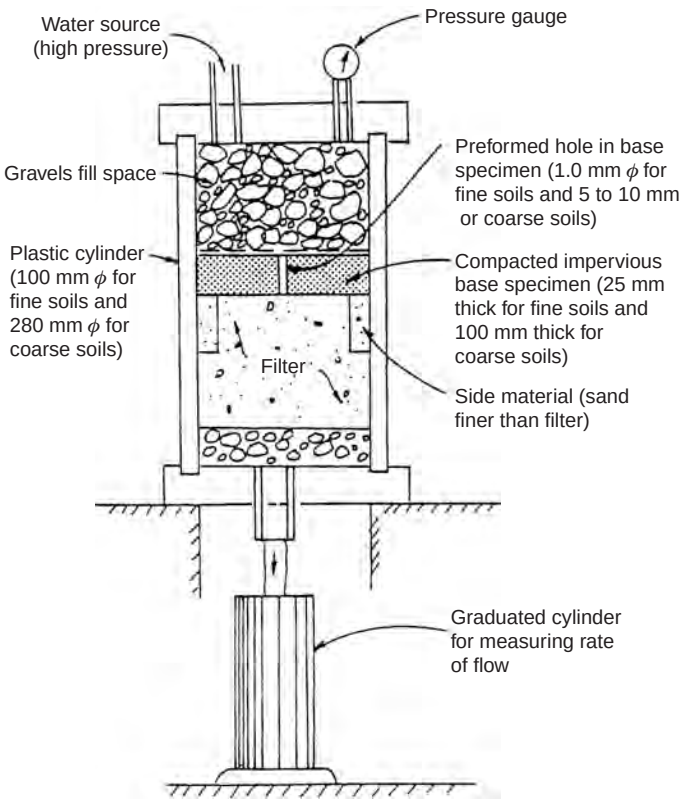

Water source
(high pressure)
Pressure gauge
Gravels fill space
Preformed hole in base specimen (1.0 mm ϕ for fine soils and 5 to 10 mm or coarse soils)
Plastic cylinder (100 mm ϕ for fine soils and 280 mm ϕ for coarse soils)
Compacted impervious base specimen (25 mm thick for fine soils and 100 mm thick for coarse soils)
Filter
Side material (sand finer than filter)
Graduated cylinder for measuring rate of flow

Figure 9.7 USSCS "no erosion" filter test apparatus (schematic, no scale) (Sherard and Dunnigan, 1985, reproduced with permission of ASCE).

but Sherard and Dunnigan (1989) found that the use of D_{50B} for their test data did not discriminate properly between tests for which the filters sealed and those which did not.

9.1.4.3 *Laboratory test equipment*

The particle size distribution of critical filters is determined using methods which are based on laboratory testing, where base soils are placed against a filter and water is passed through the sample under pressure. Figures 9.6 and 9.7 show the test apparatus used by the US Soil Conservation Service which is reported in Sherard *et al.* (1984a and b) and Sherard and Dunnigan (1985).

Sherard *et al.* (1984a and b) also used a test set up with the base soil as a slurry and slot tests.

The test shown in Figure 9.7 is intended to model the situation where a concentrated leak has formed through the dam core, so that high hydraulic heads and gradients can occur at the interface between the base soil and the filter (Figure 9.8).

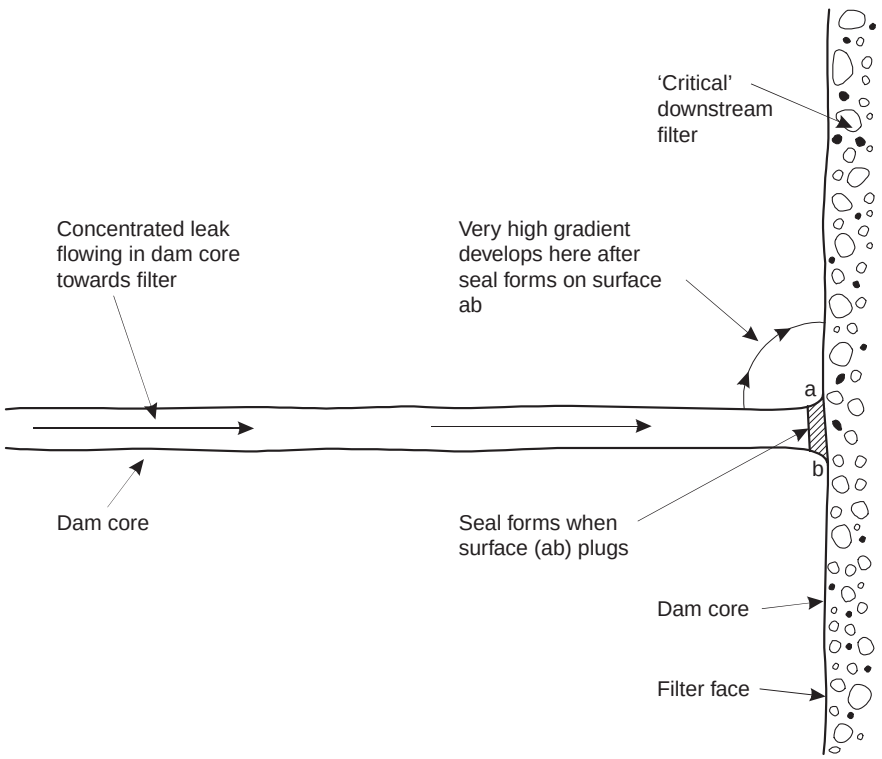

'Critical' downstream filter
Concentrated leak flowing in dam core towards filter
Very high gradient develops here after seal forms on surface ab
a
b
Dam core
Seal forms when surface (ab) plugs
Dam core
Filter face

Figure 9.8 Sketch showing concentrated leak through dam core discharging into downstream filter (no scale) (Sherard *et al.*, 1984b, reproduced with permission of ASCE).

The tests carried out by Sherard *et al.* (1984a and b) were directed towards finding the filter particle size distribution which would give no-erosion (or at least very minor erosion) conditions.

Figure 9.9 shows the type of equipment used by Bakker *et al.* (1990) to test for parallel flow conditions. Other test equipment to test this contact erosion condition is described in Section 8.6.2.

9.2 DESIGN OF CRITICAL AND NON-CRITICAL FILTERS

9.2.1 Particle size based methods for designing no erosion filters with flow normal to the filter

ICOLD (1994) describes the evolution of particle size distribution based filter design practice. These have basically evolved from the concepts of Terzaghi (1926) who proposed that $D_{15F}/D_{85B} \leq 4$ to control erosion and $D_{15F}/D_{15B} \geq 4$ to ensure the filter was sufficiently permeable.

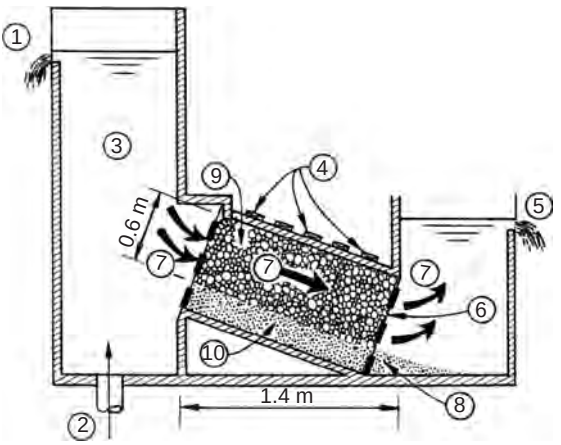


Figure 9.9 Delft Hydraulics Laboratory filter-box, with parallel flow conditions (ICOLD, 1994, from Bakker *et al*., 1990). Note: (1) Overflow, (2) water supply, (3) inflow, (4) ballast, (5) overflow, (6) screen, (7) direction of flood, (8) sand trap, (9) filter, (10) Base soil (sand).

Extensive laboratory test programs by Bertram (1940), who worked under the direction of Terzaghi and Casagrande and then by United States Bureau of Reclamation (USBR, 1955) and the US Corps of Engineers (US Corps of Engineers, 1941).

The USBR method was widely adopted for design; so many existing dams were designed using this method.

9.2.1.1 *Original USBR method*

The original USBR method as described in USBR (1977) was:

(i) (a) $D_{15F}/D_{15B} = 5$ to 40, provided that;
(b) The filter does not contain more than 5% fines passing 0.075 mm, and the fines should be cohesionless.
(ii) $D_{15F}/D_{85B} \leq 5$;
(iii) The grain size curve of the filter should be roughly parallel to that of the base material;
(iv) Maximum size particles in filter = 75 mm to prevent segregation during placement;
(v) For base materials which include gravel particles, the base material D_{15B} and D_{85B} etc. should be analysed on the basis of the gradation of the soil finer than 4.7 mm.

Criteria (ia) and (ib) are designed to ensure that the filter is more permeable than the base soil but will not hold a crack.

Criteria (ii) and (iii) are designed to ensure that the filter is sufficiently fine to control erosion of the base soil.

The authors' experience with applying this method was that, while it could be successfully applied to base soils which are clayey sands or sandy clays, it cannot be rigorously applied to base soils with a high clay and silt content. In particular:

- Rule (iii) cannot be followed (this is also the experience of Sherard and Dunnigan, 1985).
- Rule (ib) will almost always overrule (ia), often to the concern of inexperienced engineers, who feel that the "mathematic rule", i.e. (ia), should stand.

The authors' approach had been to be comfortable with rule (ib) overriding rule (ia); to ignore rule (iii) and substitute in its place the US Corps of Engineers (1941) requirement that the uniformity coefficient $D_{60F}/D_{10F} \leq 20$.

However we would not now use this method, nor does the USBR.

9.2.1.2 *Sherard and Dunnigan method*

The USSCS (United States Soil Conservation Service) (Sherard *et al.*, 1984a, 1984b and Sherard and Dunnigan, 1985) carried out extensive laboratory testing to check filter criteria. They used several different test apparatus to simulate a concentrated leak in a dam.

These included:

- Tests using the equipment shown in Figure 9.6 for base soils in the fine to coarse sand ($D_{15B} = 0.075$ mm to 2.36 mm) range.
- Tests using slot and slurry tests on silt and clay base soils, in which a small amount of erosion was accepted as indicating a successful filter.
- Tests using the "no-erosion" filter test apparatus shown in Figure 9.7. For a filter to be successful it was required that there should be no visible increase in the size of the hole in the base soil and only "very slight" erosion of the base soil.
- Sherard and Dunnigan claimed to have tested dispersive and non-dispersive silt and clay soils, but in fact they only tested two dispersive soils (Foster, 1999, Foster and Fell, 1999a).

Based on these tests, Sherard and Dunnigan (1985, 1989) recommended the following:

1. For all soils with a gravel component (except Group 3 below), the filters should be designed on the grading of that part of the soil finer than 4.75 mm.
2. *Impervious Soil Group 1* (fine silts and clays): For fine silts and clays that have more than 85% by weight of particles finer than the 0.075 mm sieve, the allowable filter for design should have $D_{15F} \leq 9D_{85B}$.
3. *Impervious Soil Group 2* (sandy silts and clays and silty and clayey sands): For sandy (and gravelly) impervious soils with 40% to 85% by weight (of the portion finer than the 4.76 mm sieve) finer than the 0.075 mm sieve, the allowable filter for design should have $D_{15F} \leq 0.7$ mm.
4. *Impervious Soil Group 3* (sands and sandy gravels with small content of fines): For silty and clayey sands and gravels with 15% or less by weight (of the portion finer than the 4.76 mm sieve) finer than the 0.075 mm sieve the allowable filter

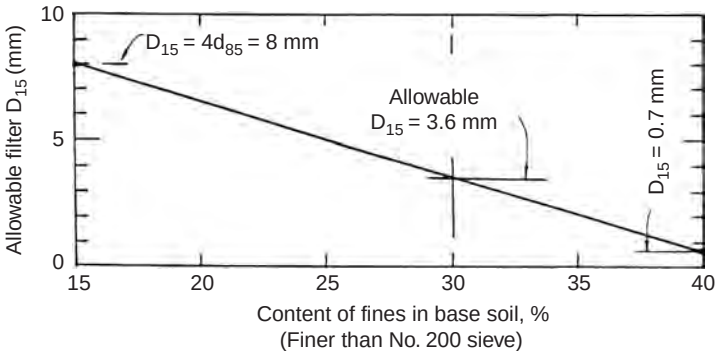


Figure 9.10 Determination of allowable D_{15F} for filters for impervious Soil Group 4 (having between 15% and 40% finer than 0.075 mm sieve) (Sherard and Dunnigan, 1985, reproduced with permission of ASCE).

for design should have $D_{15F} \leq 4D_{85B}$ where D_{85B} can be the 85% finer size of the entire material including gravels.

5. *Impervious Soil Group 4:* For coarse impervious soils intermediate between Groups 2 and 3 above, with 15% to 40% passing the 0.075 mm sieve, the allowable filter for design is intermediate, inversely related linearly with the fines content and can be computed by straight line interpolation. As an example (Figure 9.10) for an impervious sandy soil with 30% of silty or clayey fines and $D_{85B} = 2$ mm, the allowable filter for design is in between the value of $D_{15F} = 0.7$ mm (for soils of Group 2) and $D_{15F} = 4 \times (2) = 8$ mm (for soils of Group 3), and is calculated as follows:

$$D_{15F} = \frac{40 - 30}{40 - 15}(8 - 0.7) + 0.7 = 3.6\,\text{mm}$$

6. As well as having D_{15F} sizes as set out above, the filters for Soil Groups 1 and 2 must be composed wholly of sand or gravelly sand in which >60% is coarser (that is <40% finer) than 4.76 mm and the maximum particle size is 50 mm.
7. The above criteria can be applied for all soils in Groups 1 and 2 regardless of the shape of the particle size distribution curve. For soils of Groups 3 and 4 the criteria apply to reasonably well-graded soils. For soils in Groups 3 and 4 which are highly gap-graded it is desirable to provide a filter for the finer portion of the gap-graded soil, or carry out No Erosion Filter Tests in the laboratory to select the appropriate filter.

Sherard and Dunnigan (1985, 1989) indicate that these criteria already incorporate an adequate factor of safety.

The criteria listed above do not include specific reference to limiting the "fines" content, i.e. silt and clay passing 0.075 mm sieve, except that (6) indicates filters for Soil Groups 1 and 2 should be composed "wholly of sand and gravel". Some limitation on fines content and nature such as that required by the USBR (1977) seems to be

Table 9.2 Summary of statistical analysis of the results of no-erosion filter tests, and proposed no-erosion filter criteria (Foster, 1999; Foster and Fell, 1999a, 2001).

Base soil group	*Fines content (a)*	*Design criteria of Sherard and Dunnigan (1989)*	*Range of DF15 for no erosion boundary D_{85B}*	*Criteria for no erosion boundary*
1	≥85%	$D_{15F} \leq 9D_{85B}$	$6.4–13.5D_{85B}$	$D_{15F} \leq 9D_{85B}$ (b)
2A	35–85%	$D_{15F} \leq 0.7$ mm	0.7–1.7 mm	$D_{15F} \leq 0.7$ mm (b)
3	<15%	$D_{15F} \leq 4D_{85B}$	$6.8–10D_{85B}$	$D_{15F} \leq 7D_{85B}$
4A	15–35%	$D_{15F} \leq$ (40–pp% 0.075 mm)× $(4D_{85B}–0.7)/25 + 0.7$ mm	$1.6D_{15F}–2.5D_{15F}$ of Sherard and Dunnigan design criteria	$D_{15F} \leq 1.6D_{15F}d$, where $D_{15F}d =$ (35–pp% 0.075 mm) $(4D_{85B}–0.7)/20 + 0.7$ mm

Notes: (a) The subdivision for Soil Group 2 and 4 was modified from 40% passing 0.075 mm, as recommended by Sherard and Dunnigan (1989), to 35% based on the analysis of the filter test data. The modified soil groups are termed Group 2A and 4A. The fines content is the % finer than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.
(b) For highly dispersive soils (Pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower D_{15F} for the no erosion boundary: For Soil Group 1, use the lower limit of the experimental boundary, i.e. $D_{15F} \leq 6.4D_{85B}$. For Soil Group 2A, use $D_{15F} \leq 0.5$ mm.

implied i.e. filters should contain not more than 5% fines passing 0.075 mm, and the fines should be "cohesionless".

It should be noted that for Soil Group 1 hydrometer particle size analysis will be required to define the particle size below 0.075 mm.

The Sherard and Dunnigan (1985, 1989) method has become widely adopted including by USBR (1989a), USDA-SCS (1994), FEMA (2011) and throughout Australia.

9.2.1.3 *Foster and Fell method*

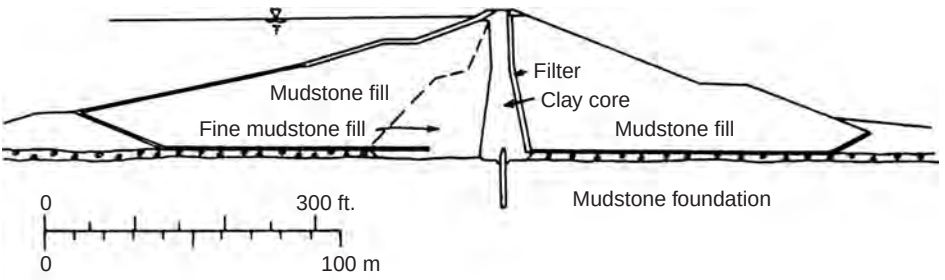
Foster (1999) and Foster and Fell (1999a) carried out extensive no-erosion tests using a test set-up similar to that of Sherard *et al.* (1984a,b), and reviewed the results of the USSCS tests as reported in internal reports by Sherard.

Table 9.2 summarizes the results. The "criteria for no-erosion boundary" are intended to apply to the assessment of filter performance of existing dams and do not include any margin of safety.

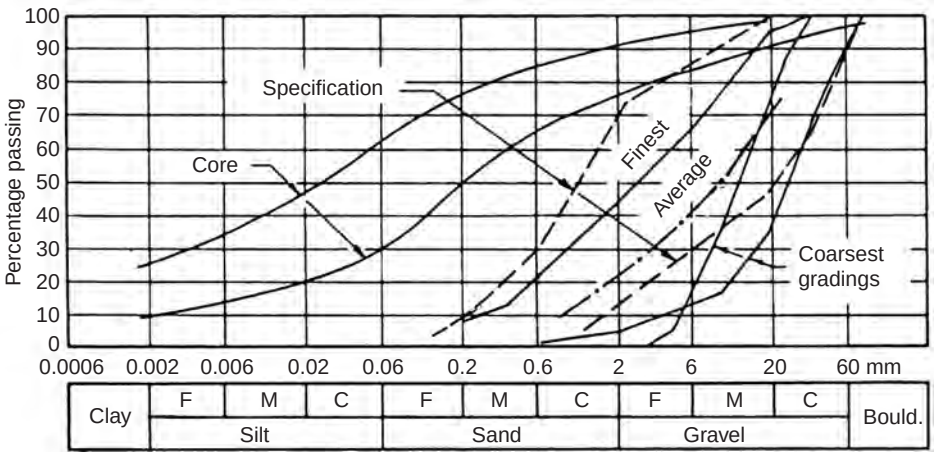
This work showed that the division between Group 2 and 4 soils is better defined on a fines content of 35% than on the 40% used by Sherard *et al.* (1984a, b) and that the Sherard and Dunnigan (1989) criteria generally have a margin of safety but they are not sufficiently conservative to define no-erosion condition for dispersive soils.

9.2.1.4 *Vaughan and Soares method*

Vaughan and Soares (1982) provoked considerable discussion when, in a review of partial piping failure of Balderhead Dam in northern England, they claimed that in some base soils self filtering could not be relied upon and that it was necessary to design filters to prevent the passing of clay flocs into the filter.



Balderhead dam: Cross-section



Balderhead dam: Core and filter gradings

Figure 9.11 Balderhead Dam core and filter gradings (Vaughan and Soares, 1982).

The core for the Balderhead Dam had been constructed from well graded glacial till, which had a high resistance to erosion. Filters were crusher run hard limestone with grading limits shown on Figure 9.11.

Cracks occurred in the dam core by hydraulic fracture and Vaughan and Soares (1982) postulated that the partial piping failure, which occurred more than a year after the dam was filled, was due to erosion of the finer clay particles in the cracks. The flow velocity in the cracks was postulated to be too low to transport coarser particles from the core, so preventing self filtering within the core material and allowing the clay particles to erode into the filter. As discussed above, Kenney *et al.* (1985) observed similar behaviour in two dams constructed of widely graded glacial till.

To overcome this, Vaughan and Soares (1982) proposed the concept of a "perfect filter". In this approach, the particle size distribution of the clay core is obtained in a hydrometer test, but with no dispersant added. The water used for the test should be of the same chemistry as the water to be seeping through the dam since the floc size (degree of dispersion) is dependent on the water chemistry.

Having determined the particle size distribution, a sample of the floc sized sediment is prepared by sedimentation and used in filter experiments. In these experiments the flocs are introduced to the filter in a dilute suspension.

Tests carried out in this manner on soils from two dams indicated the need for a significant silt fraction in the filter and that even a fine to medium sand size filter may not be satisfactory,

The general tone of the discussion which followed the paper was that "based on experience with other laboratory experiments and performance of dams" the Vaughan and Soares method was too conservative. Vaughan and Soares in the closure discussion stated "the writers would stress that the need to meet these criteria completely is one for the designer of any particular dam to assess, along with other factors involved in the provision of effective filters".

The "perfect filter" method will often give finer filters than particle size based methods because it does not rely on self filtering and the authors are not aware of any general acceptance of the concept. It will sometimes require a three stage filter whereas other methods may require only two.

Vaughan and Soares' case is weakened somewhat by the coarse grading of some of the filters as constructed for Balderhead Dam – these would not meet USBR (1977) or Sherard and Dunnigan (1985) criteria.

9.2.2 Methods based on constriction or opening size

There are a number of these methods including Brauns and Witt (1987); Schuler and Brauns (1993, 1997); Lafleur *et al.* (1993); Giroud (1996), Indraratna and Raut (2006); Indraratna *et al.* (2007) and Raut and Indraratna (2008).

Generally, these methods relate the controlling constriction size which is a size characteristic of the void network in a granular filter and is equal to the diameter of the largest particle that can possibly be transported through the filter by seepage to a % of fine passing sizes.

Most relate constriction size to the $D_{15F.}$ e.g. Kenny *et al.* (1985) used:

$$D_C^* \leq 0.25D_{5F} \tag{9.1a}$$

and

$$D_C^* \leq 0.20D_{15F} \tag{9.1b}$$

Indraratna *et al.* (2007) and Raut & Indraratna (2008) propose a constriction based filter design criteria:

$$D_{c35}/d_{85}^* \leq 1$$

where D_{c35} is the controlling constriction size of the filter, and d_{85}^* is the d_{85} of the modified (regraded) base soil by neglecting the base soil particles larger than the self-filtering constriction size D_{c95}. The method for estimating the constriction size is described in Indraratna *et al.* (2007) and takes account of the particle size distribution of the filter and its relative density which influences the constriction size.

The authors see the merit of using constriction size because logically that is what controls the movement of the base soil into the filter. However we have not used these methods because they require particle size distributions as inputs to the methods and we can see little advantage in converting these to constriction size. In our experience specifications for construction of filters are always expressed in terms of particle size distribution.

The methods of Indraratna and his co-workers require iterative calculations to determine the constriction size and are as a result more difficult to apply than particle size based methods.

9.2.3 Methods based on the permeability of the filter

9.2.3.1 *Delgardo and co-workers*

Delgado *et al.* (2006) carried out extensive filter tests to determine no-erosion filter (NEF) criteria based on the percentage of the base soil passing the 0.075 mm sieve, and the permeability of the filter.

They combined these data with that of Sherard *et al.* (1984), Foster and Fell (1999a) and others. For the University of Granada (UGR) tests Delgado *et al.* (2006) determined the permeability by laboratory tests, thereby taking account of the degree of compaction of the filter.

For the tests done elsewhere Delgado *et al.* (2006) determined the filter permeability using the relationship from Sherard and Dunnigan (1989) ($k = 0.35(D_{15})^2$, where (D_{15}) is in mm, and k is in cm/sec).

9.2.3.2 *Vaughan and Soares, Vaughan and Bridle method*

Vaughan and Soares (1982), Vaughan and Bridle (2004) also carried out a series of tests to determine the sizes of floc trapped by filters of varying permeability. They developed an equation linking permeability of filter to the floc size that the filter will trap.

The authors do not recommend the use of permeability based methods. The reasons for this are:

(a) The permeability does not control the erosion of particles; it is the constriction or opening size of the filter and the particle size of the base soil. The constriction size is directly related to particle size distribution but only indirectly related to permeability.
(b) The testing of permeability in the laboratory is more difficult and time consuming than testing of particle size distribution. Repeatability of tests is difficult and it would be a source of contractual dispute if permeability was specified as the primary control for filters.
(c) As the Delgardo *et al.* (2006) method includes the tests of Sherard *et al.* (1984a, b) and Foster and Fell (1999a) it is not likely to give significantly different filter designs than those methods which are widely used and easy to apply.

Table 9.3 Base soil categories.

Base soil category	*% finer than 0.075 mm (after regrading, where applicable)*	*Base soil description*
1	>85	Fine silts and clays
2A	35–85	Silty and clayey sands; sandy clays; and clay, silt, sand, gravel mixes
4A	15–35	Silty and clayey sands and gravel
3	<15	Sands and gravel

9.2.4 Recommended method for design of critical no erosion filters, with flow normal to the filter

The recommended method for design of critical no-erosion filters with flow normal to the filter (flow types N1 and N2), is based on Sherard and Dunnigan (1985, 1989). They however did not give explicit instructions on how to apply their criteria. This has been better detailed in USBR (1989a) and USDA-SCS (1994).

The following is taken largely from the US Soil Conservations Service, USDA-SCS (1994). It has been modified to include the changes to the Base Soil Group boundaries and for dispersive soils proposed by Foster (1999) and Foster and Fell (1999a, 2001).

The base Soil Groups are re-named 1, 2A, 3A and 4 to distinguish them from those used by Sherard and Dunnigan (1985, 1989). The method is to determine filter gradation limits using the following steps:

Step 1: Plot the gradation curve (grain-size distribution) of the base soil materials. Use enough samples to define the range of grain sizes for the base soil or soils. Design the filter using the base soil that requires the smallest D_{15F} size for filtering purposes. Base the design for drainage purposes on the base soil that has a representative (say median) D_{15B} size.

Step 2: Proceed to Step 4 if the base soil contains no gravel (material larger than 4.75 mm) or if designing coarse filters, where the base soil is the fine filter.

Step 3: Prepare adjusted gradation curves for base soils that have particles larger than the 4.75 mm sieve.

- Obtain a correction factor by dividing 100 by the percent passing the 4.75 mm sieve.
- Multiply the percentage passing each sieve size of the base soil smaller than 4.75 mm sieve by the correction factor determined above.
- Plot these adjusted percentages to obtain a new gradation curve.
- Use the adjusted curve to determine the percentage passing the 0.075 mm sieve in Step 4.

Step 4: Place the base soil in a category determined by the percent passing the 0.075 mm sieve from the regraded gradation curve data according to Table 9.3.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15F} size for the filter in accordance with the Table 9.4.

Table 9.4 Filtering criteria for critical filters – maximum D_{15F}.

Base soil category	*Filtering criteria*
I	$\leq 9 \times d_{85B}$ but not less than 0.2 mm (a)
2A	$\leq$0.7 mm (b)
3	$\leq 4 \times d_{85B}$ of base soil after regrading
4A	$(35-A)/(35-15)[(4 \times D_{85B}) - 0.7\text{ mm}] + 0.7\text{ mm}$ A = % passing 0.075 mm sieve after regrading (If $4 \times D_{85B}$ is less than 0.7 mm, use 0.7 mm)

Notes: (a) $\leq 6D_{85B}$ for dispersive soils. (b) $\leq$0.5 mm for dispersive soils.
Dispersive soils are soils with pinhole classification D1 or D2, or Emerson class 1 or 2.

Table 9.5 Permeability criteria.

Base soil category	*Criteria*
All categories	Minimum $d_{15F} \geq 4 \times D_{15B}$ of the base soil before regrading, but not less than 0.1 mm $\leq$2% (or at most 5%) fines passing 0.075 mm sieve in the filter; fines non plastic

Table 9.6 Other filter design criteria.

Design element	*Criteria*
To prevent gap-graded filters Filter band limits	The width of the designed filter band should be such that the ratio of the maximum diameter to the minimum diameter at any given percent passing value $\leq$60% is $\leq$5 Coarse and fine limits of a filter band should each have a coefficient of uniformity of 6 or less

Step 6: To ensure the filter is sufficiently permeable, determine the minimum allowable D_{15F} in accordance with Table 9.5. The permeability requirement is determined from the D_{15} size of the base soil gradation before regrading.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters, but wide enough to allow manufacture. Adjust the maximum and minimum D_{15F} sizes for the filter band determined in Steps 5 and 6 so that the ratio is 5 or less at any given percentage passing of 60 or less and adjust the limits of the design filter band so that the coarse and fine sides have a coefficient of uniformity D_{60}/D_{10} of 6 or less. Criteria are summarized in Table 9.6. This step is required to avoid the use of gap-graded filters. The use of a broad range of particle sizes to specify a filter gradation could result in allowing the use of gap-graded materials. Materials that have a broad range of particle sizes may also be susceptible to segregation during placement.

Step 8: To minimize segregation during construction, use a maximum size of 75 mm in filter zones which are not less than 2 m wide or 0.5 m thick. For narrower and thinner filter zones (particularly Zone 2A filters) use a maximum size of 37 mm or 50 mm. Consider the relationship between the maximum D_{90} and the minimum D_{10} of the filter. Calculate a preliminary D_{10F} size by dividing the minimum D_{15F} by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15F} and D_{10F} should be on a coefficient of uniformity of about 6.)

Table 9.7 Segregation criteria.

Base soil category	*If D_{10F} (mm) is*	*Then maximum D_{90F} (mm) is*
All categories	<0.5	20
	0.5–1.0	25
	1.0–2.0	30
	2.0–5.0	40
	5.0–10	50
	>10	60

Determine the maximum D_{90F} using Table 9.7. For Zone ZB filters, use the coarse limit D_{10F} in Table 9.7.

Sand filters that have a D_{90F} less than about 20 mm generally do not require special adjustments for the broadness of the filter band. For coarser filters and gravel zones that serve both as filters and drains, the ratio of D_{90F}/D_{10F} should decrease rapidly with increasing D_{10F} sizes.

Step 9: Connect the control points to form a preliminary design for the fine and coarse sides of the filter band. Complete the design by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values. Designers will find it valuable to have the selected design limits plotted on a drawing for use in construction.

For situations where filters close to the allowable limits are being contemplated, or being checked in an existing dam, it is recommended that no-erosion tests as described by Sherard *et al.* (1984b) be carried out. This is particularly important for fine-grained soils, where the separation of the no-erosion and continuing erosion boundaries is small (see Section 9.3) and for dispersive soils, because of the limited amount of testing upon which the design criteria are based.

9.2.5 Recommended method for design of less critical and non-critical filters

9.2.5.1 *Filters upstream of the dam core*

The filter upstream of a central core earth and rockfill dam (Zone 2C) is not subject to continuous seepage exit gradients or to the risk of high exit gradients if the core cracks.

Zone 2C filters can therefore be designed less conservatively than Zone 2A and 2B filters. It is common only to require that "*Zone 2C filters shall be constructed of well graded rock-fill with a maximum size of 150 mm*". This will result in most cases in a sand/gravel/cobble size mixture of rock fragments, which provides a transition between the earthfill core and the upstream rockfill. Since this zone will be won from the quarry (probably after passing through a "grizzly" to remove oversize rock) it is relatively inexpensive and it would be normal to place at least 4 m width, possibly 6 m on a larger dam. The added width is important so as to counter the effects of likely segregation.

However, care should be taken in adopting this approach. A more robust and defensible approach, which the authors would recommend, would be to accept a relaxed specification for these filters, but to ensure they are finer than that required to satisfy the excessive (or, at worst, continuing) erosion criteria (see Chapter 8 for definitions of excessive and continuing erosion). Be aware that using filter/transition materials with rocks up to 150 mm is a recipe for segregation problems and it would be better to limit the size to a maximum of 75 mm.

Even if a relaxed specification is adopted, the fines content (passing 0.075 mm) should not exceed 5% and the fines should be non plastic.

For any dam, including pumped storage dams, where the drawdown could be as high as several metres within hours, or on a daily or even twice daily cycle, it would be wise to use conventional filters designed according to Section 9.2.4, at least in the zone of daily drawdown.

9.2.5.2 *Filters under rip-rap*

There are two requirements for these filters:

- That they are coarse enough not to wash out of the rip-rap.
- They are fine enough to prevent erosion of the soil beneath the filter.

Two layers of filters may be required to satisfy these requirements. If proper protection is required, no-erosion filter criteria should be satisfied. This is recommended by US Corps of Engineers (1984a).

It is fairly common to use more relaxed criteria in non-critical conditions where some damage can be tolerated. ICOLD (1993, 1994) suggest that, for bedding filter under rip rap, filters which are reasonably well graded, between a maximum of 80–100 mm and coarse sand sizes, are satisfactory for the great majority of dams, but warn that segregation will be a problem and that such filters will not protect against frequent surging from wave action.

The authors' view is that the design of these Zone 2D filters (Figure 13.5a and c) is not as critical as for Zone 2A filters in that, if damage does occur, it can usually be repaired and so a relaxation of the strict no erosion rules may be appropriate. If there is a reasonably well-graded sandy gravel/gravelly sand from 0.075 mm to 50 mm or 75 mm available either naturally or with a minimum of processing, this should be satisfactory in most cases. This is also the experience of Sherard *et al.* (1963). Filters which consist only of fine or medium (or even coarse) sand are unlikely to be satisfactory as they will not self-filter against the rip-rap.

The authors take the view that for thin layers of rip-rap, on important dams, with significant wave action, properly designed filters should be provided. Advantage may be taken of the knowledge of the work described above, or simply standard no-erosion filters designed using Section 9.2.4.

For smaller dams, where the risk of deterioration and the need for repairs may be acceptable, geotextiles may be used under rip-rap. If so, the following points should be noted:

(a) Use non-woven needle punched heavy quality fabric, e.g. Bidim A44.

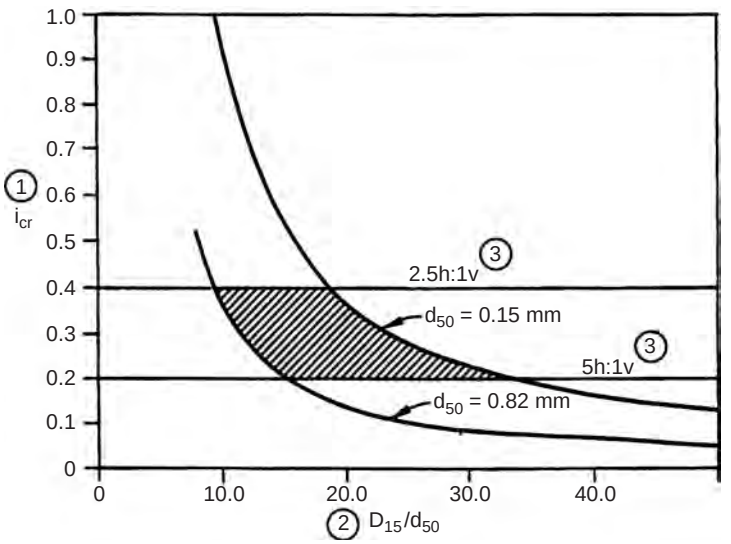


Figure 9.12 Critical gradient with steady flow parallel to the interface (ICOLD, 1994, based on Bakker, 1987).

(b) Provide a bedding layer beneath the rip-rap of gravel or gravel-cobble mix directly onto the fabric to avoid developing folds in the fabric between the particles, below which soil could be eroded.

(c) Take account of the potentially low friction coefficient between the geotextile and the underlying soil as discussed in Chapter 13.

9.2.6 Review of available methods for designing filters with flow parallel to the filter

ICOLD (1994) summarize testing at Delft Hydraulic Laboratory (Bakker, 1987) to test the condition where flow in the filter is along, or parallel to, the filter. Den-Adel *et al.* (1994) provide details of a method for designing such filters.

It is apparent that in these situations, erosion is less likely and considerably coarser filters than obtained from the criteria discussed in Sections 9.1 and 9.2.4 can be used.

Figure 9.12 summarizes some of the Delft Laboratory Testing.

In this figure the hydraulic gradient, i_{cr}, e.g. equal to the slope of the filter under rip-rap under the upstream face of a dam, is plotted against D_{15F}/D_{50B} (D_{15}/d_{50} in Figure 9.12). Curves for sandy base soils with $D_{50} = 0.15$ mm and 0.82 mm are shown. Thus for a uniform sand base material with a d_{50} of 0.15 mm on a slope of 2.5H:1V ($i_{cr} = 0.4$), the ratio of D_{15F}/D_{50B} can be as high as about 8.

Great care should be taken in applying these approaches, particularly to situations where, if the filter fails and erosion occurs, it is not practical to repair the damage. The authors would use conventional no-erosion filters for such new structures.

Table 9.8 USDA-SCS (1984) criteria for filters adjacent to perforated collector pipes.

Situation	*Criteria*
Non critical drains where surging or gradient reversal is not anticipated	$D_{85F} \geq$ perforation size
Critical drains where surging or gradient reversal is anticipated	$D_{15F} \geq$ perforation size

9.2.7 Design criteria for pipe drains and pressure relief well screens

9.2.7.1 *Pipe drains*

The authors do not favour the use of pipe drains to increase the discharge capacity of filter/drains, because they are susceptible to corrosion and blocking and joints may open with differential movement in the dam and the foundation, allowing the surrounding filter material to enter the pipe and block it.

However many dam engineers prefer to install pipes particularly for drains sited at the downstream toe of the embankment where they can be accessed for repair if necessary.

For these and the assessment of existing installations, there are several methods available:

(a) USDA-SCS (1994) whose requirements as shown in Table 9.8.
(b) USBR (1989a) who recommended criteria $D_{85F} \geq 2 \times$ (perforation size).
(c) FEMA (2011) who recommend that the maximum pipe perforation dimension should be no larger than the finer side of the D_{50E} where D_{50E} is taken from the gradation of the envelope (drain) material that surrounds the drainpipe.

As the opening size $O_E = D_{15F}/9$ the USDA-SCS requirement for non-critical drains is likely to result in some erosion into the drain.

The authors suggest that for design of drains and the surrounding filter $D_{85F} \geq 2 \times$ (perforation size) is adopted. This assumes that the drain material has a uniformity coefficient 5 or less. For drains where surging or gradient reversal is anticipated $D_{15F} \geq$ (perforation size) is suggested as recommended by USDA-SCS.

As recommended by FEMA (2011) two stage filter drains are recommended as shown in Figure 9.13. If this is done and a sizeable zone of drain material is provided the pipe drain is in any case seldom necessary from a discharge viewpoint.

FEMA (2011) point out that collector pipes have a long history of clogging and collapse. They recommend that profile HDPE pipe be used in toe drains. The advantages of this pipe type over all others are large load carrying capability; the pipe is more likely to withstand poor or incorrect installation methods; joints are welded, strong and water tight; junctions are factory welded, strong and water tight; aftermarket perforations can be used allowing the designer to specify the perforation size permitting more flexibility n the selection of gravel envelope material.

FEMA (2011) recommend that perforated collector pipes should always be inspected video camera at the end of construction to verify no damage occurred during installation.

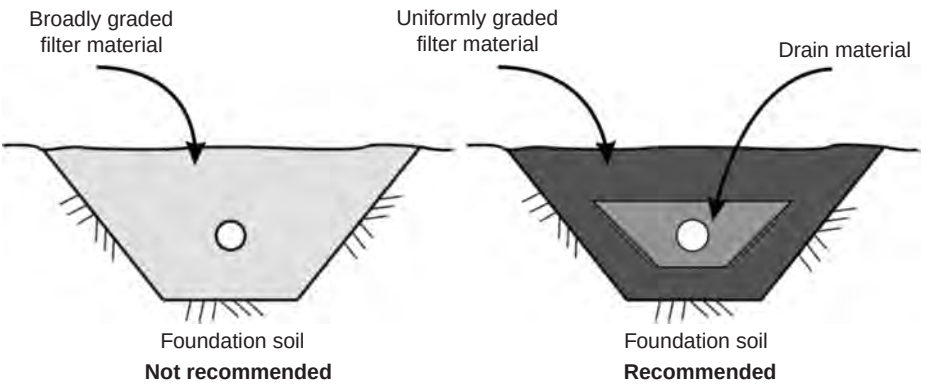


Figure 9.13 Recommended and not recommended toe drain design details (FEMA, 2011).

As stated above the authors do not favour the use of pipe drains and would certainly not place them beneath the embankment. If they are to be used they should be constructed with properly designed filters and so they can be inspected and cleaned if blocked.

9.2.7.2 *Pressure relief well screens*

Relief wells are usually constructed with well screens, much like a water well, with an annular space surrounding the well screen containing a designed filter pack. If the soil surrounding the relief well is very fine a two-stage filter pack may be required.

FEMA (2011) refer to USACE EM 1100-2-1913 which recommends that the maximum screen opening dimension should be no larger than the finer side of the D_{50E} where D_{50E} is the gradation of the filter.

Bird and Davidson (2013) indicate that in their experience this works unless the filter is very uniform, in which case they use a finer opening screen. They require that the allowable amount of filter washing in to the well be less than 1 or 2 parts per million of the water flow where soil loss over time would be detrimental to the performance of the structure.

Hadj-Hamou *et al.* (1990) carried out tests on well screens and found that for the filter/screen combinations they tested quite large amounts of filter loss occurred during initial surging (development) of the well designed on D50E but the filter function was judged to be unaffected. They found less filter loss with screens openings of D20E and D10E. The latter test was for a uniform filter. They point out that finer screens are more likely to block with time.

It is concluded from this that:

(a) Design screen openings no larger than D_{50E} where D_{50E} is the finer side of the gradation of the filter.
(b) For uniform filters design screen openings no larger than D_{15E}. This is selected for consistency with the requirements for design of pipe drains.

(c) Monitor the development of the wells to confirm that the amount of filter washing into the well is less than 1 or 2 parts per million of the water flow where on-going erosion will be detrimental to the operation of the well or the structure.

Further discussion on the design of pressure relief wells is given in Section 10.7.4.

9.2.8 Other factors affecting filter design and performance

9.2.8.1 *Criteria to assess internal instability or suffusion*

Filters which are gap graded or with a flat "tail" in the finer fraction as shown in Figure 9.5 may be internally unstable and subject to suffusion. If this occurs the finer fraction may partly or wholly erode resulting in a coarser filter.

Methods for assessing the likelihood of internal instability are discussed in Section 8.5. For gap graded filters the Kenny and Lau (1985a, b) and Wan and Fell (2004c, 2008) modified Burenkova (1993) method are applicable. For filters with a flat "tail" the Wan and Fell (2004c, 2008) modified Burenkova (1993) method is applicable. For filters with considerable finer fraction the Kenny and Lau (1985a, b) method is likely to be conservative.

9.2.8.2 *Segregation*

Characteristics which make sand-gravel filter segregate on placement include:

- A broad grading, particularly with maximum particle size >75 mm.
- A low percentage of sand and fine gravel sizes (<40% finer than 4.75 mm).
- Poor construction practices, e.g. end dumping from trucks, high lift heights and poor control of stockpiling operations.

Sherard and Dunnigan (1989) suggested use of filters with a maximum size not greater than 50 mm and required no less than 40% sand and fine gravel passing 4.75 mm. Ripley (1986) recommended for filters in contact with the core:

- Maximum size 19 mm.
- Not less than 60% finer than 4.75 mm.
- Not greater than 2% finer than 0.075 mm.

These criteria were based on construction experience.
Kenney and Westland (1993) carried out laboratory tests and concluded that:

- All dry soils consisting of sands and gravels segregate in the same general way, independent of grain size and grain size distribution.
- Dry soils containing fines <0.075 mm segregate to a smaller extent than soils not containing fines.
- Water in sandy soils (mean size finer than 3 mm to 4 mm) inhibits segregation but has little influence on the segregation of gravels (mean size coarser than 10 mm to 12 mm).

The USDA-SCS (1994) and USBR (1989a) have adopted a maximum size of 75 mm and limits on the minimum D_{10F} and maximum D_{90F} to limit segregation. These are

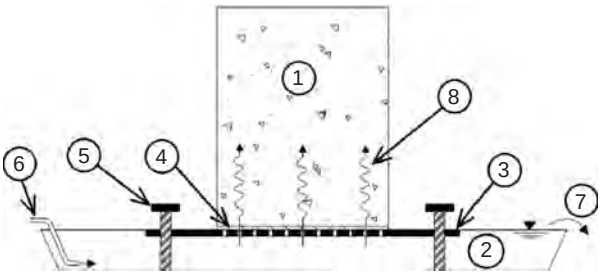


Figure 9.14 Schematic illustration of test set up for sand castle test. 1: Filter specimen, 2: water tray, 3: perforated base plate, 4: wire screen, 5: levelling screw, 6: inflow hose, 7: overflow, 8: water capillary rise (Soroush *et al.*, 2012).

described in Section 9.2.4. The authors believe that for narrow or thin filter zones, 75 mm may be too large and recommend the use of maximum size of 37 mm or 50 mm in these situations.

9.2.8.3 *Ability of the filter to hold a crack*

It is unfortunately not uncommon to be presented with information showing that the filters or transition zone in an existing dam were constructed with fines content (% passing 0.075 mm) greater than the 5% normally accepted as an upper limit and/or that the fines are, contrary to accepted practice, plastic.

The question which arises is whether the filter or transition will "hold a crack", and not perform its filter function. ICOLD (1994) describe a simple test attributed to Vaughan for assessing this.

> "*A simple test, suitable for use in a field laboratory, has been devised to examine filter cohesion. It consists of forming a cylindrical or conical sample of moist compacted filter, either in a compaction mould or in a small bucket such as is used by a child on a beach; standing the sample in a shallow tray (if a bucket is used the operation is exactly as building a child's sand castle) and carefully flooding the tray with water. If the sample then collapses to its true angle of repose as the water rises and destroys the capillary suctions in the filter, then the filter is non-cohesive. Samples can be stored for varying periods to see if cohesive bonds form with time. This test is, in effect, a compression test performed at zero effective confining pressure and a very small shear stress, and it is a very sensitive detector of a small degree of cohesion.*"

Refinements of this test have been proposed by Soroush *et al.* (2012). They used the test set up shown in Figure 9.14. The sample of the filter is compacted into a 155 mm diameter 200 mm high split mould at the relative density and moisture content the filters are placed in the embankment. The sample is removed from the mould and placed upon a perforated base plate then placed in water filled tray with the water flush with the surface of the perforated plate. This prevents undermining of the sample while allowing capillary rise.

Table 9.9 Likelihood of filters holding a crack based on sand castle test (Soroush *et al.*, 2012).

Test collapse time (minutes)[a,b]	*Likelihood of the filter holding a crack*
≤ 2	Very unlikely
$2 \leq 5$	Unlikely
$5 \leq 15$	Unlikely to likely
$15 \leq 60$	Likely
>60	Almost certain

Notes: (a) Given that the specimen collapses in a bursting type of failure, which is rapid when set in motion and the specimen splits apart.
(b) At least three trials should be performed.
(c) Depending on the plasticity index.

Table 9.10 Likelihood for filters with excessive fines holding a crack (Fell *et al.*, 2008).

	Fines Content	*Probability of holding a crack*	
Fines Plasticity	*% Passing 0.075 mm*	*Compacted*	*Not compacted*
Non plastic (and no cementing present)	5%	0.001	0.0002
	7%	0.005	0.001
	12%	0.05	0.01
	15%	0.1	0.02
	>30%	0.5	0.1
Plastic (or fines susceptible to cementing)	5%	0.05	0.02
	7%	0.1	0.05
	12%	0.5	0.3
	≥15%	0.9	0.7

Note: Fines susceptible to cementing for filters having a matrix predominately of sand sized particles (e.g. filters derived from crushed limestone).

They tested non-plastic filters with a range of fines contents from 5% to 15%, and one plastic filter. From these data, and the data from Foster and Fell (1999b) and Fell *et al.* (2008) they proposed the outcomes as detailed in Table 9.9.

Rinehart *et al.* (2012) have developed a "modified sand castle test" for checking the cementation potential of filters. Their emphasis was on filters which may dry by desiccation. The procedure involves compacting the filter into a mould in a saturated condition, then drying the sample to a constant mass and incrementally wetting the sample in a water container and recording the time for collapse. They tested 16 filter materials which had a range of collapse times and proposed acceptance criteria based on their judgement.

The authors feel that these two refinements are worthy and suggest that both methods be used. More work is required to calibrate performance against the test results but the tests can be used qualitatively.

Fell *et al.* (2008) proposed Table 9.10 based on case data in Foster and Fell (1999) where piping had occurred in glacial till cores, testing by Park (2003) and their judgement. The authors still feel this gives reasonable guidance but note that "non-plastic" means just that as recent tests on filters with 8% and 12% fines, with plasticity indices

of 2% to 4% using the Soroush *et al.* (2012) method all failed to collapse and the times for collapse by the Rinehart *et al.* (2012) method were also long.

Our judgement is that it is the combination of fines content, the plasticity of the fines and degree of compaction which is important.

9.2.8.4 *Permeability*

The filter must be sufficiently permeable for the seepage flow to pass through it without significant build up of pressure. This has been taken into account by using the criteria $D_{15F}/D_{15B} > 4$ or 5, which ensures that the permeability of the filter is 15 to 20 times that of the soil. However just as important is to keep the fines content (silt and clay sized particles) to a minimum as is demonstrated in Figure 9.15 which shows the influence on permeability of the type and amount of fines.

Some, e.g. USDA-SCS (1994), USBR (1989a), allow 5% (non plastic) fines, but as is evident from Figure 9.15, this may reduce the permeability by one or two orders of magnitude compared to clean filter materials.

The authors' preference for filters which are required to act as a drain is to specify not greater than 2% or 3% fines, and that the fines be non plastic. For other fine filters up to 5% may be accepted but less is preferred. The cost in washing to achieve not more than 2% fines is not high and generally worthwhile. The second advantage of low fines content is that the filters are unlikely to hold a crack. These percentages are specified for the in place material in the embankment; the material as supplied to the site may need to be specified at a lesser % fines to allow for breakdown during handling on site and placement.

Note that the discharge capacity of a filter drain system is a separate issue, discussed in Chapter 10.

9.2.8.5 *"Blow-out" or "heave" of the filter*

In cases where there is limited depth of cover over the filter/transition zone there is potential for the filter being lifted from the soil/filter interface. In this case the filter will be ineffective. This is of particular concern for concentrated leak erosion where full reservoir head may be present at this interface as the filter seals with soil eroded from the sides of the defect or flaw.

The potential for blow out is assessed by comparing the seepage head at the downstream face of the core to the weight of soil cover. This is calculated as the ratio of the total stress from the vertical depth of soil (and rockfill) over the crack exit to the potential reservoir head.

The authors approach has been to accept that if the factor of safety is greater than about 0.5 three dimensional effects will be sufficient to make this a non-issue. If the factor of safety is less than about 0.1 it should be assumed the filter/transition will not be effective.

Alternatively the authors have been satisfied that if the factor of safety is 1.0 for reservoir full supply level, it will be sufficient at dam crest level because of 3 dimensional effects.

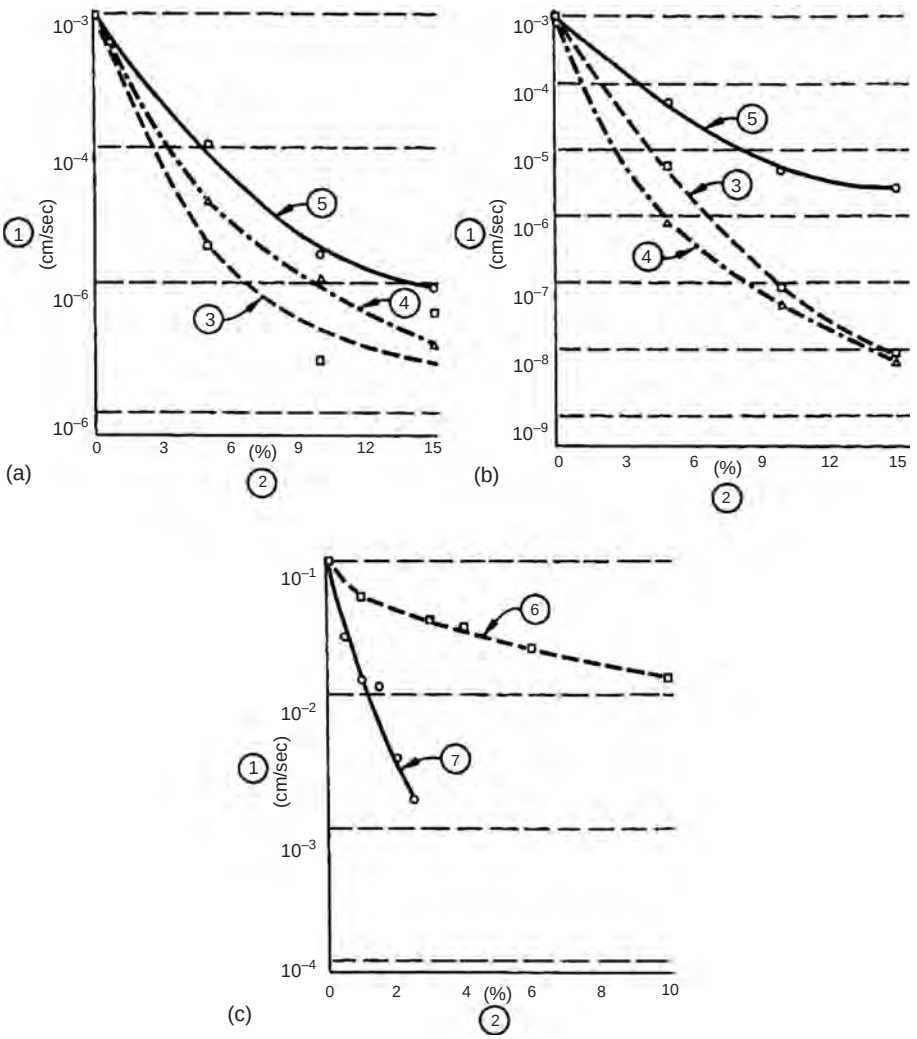


Figure 9.15 Influence of the type and amount of fines on permeability of concrete sand, sand-gravel mixture and uniform fine sand (ICOLD, 1994, from US Corps of Engineers, 1986).

9.3 ASSESSING FILTERS AND TRANSITION ZONES IN EXISTING DAMS

9.3.1 Some general issues and concepts

Many existing dams have no filters, or have filters or/and transition zones which do not satisfy no-erosion filter criteria. Foster (1999) and Foster and Fell (1999a, 2001) showed that many such filters will after some erosion of the soil being filtered eventually seal with medium size particles eroded from the protected soil so preventing the breach of the dam.

This section describes the methods which are available to allow this to be assessed. The methods are best used in a risk-based framework in which case the whole of the piping process – initiation, continuation, progression to form a pipe and breach is considered. This is discussed in Chapter 10. This section considers only the filters or transition zones, which mainly affect the continuation of erosion.

The starting point for assessing existing dams for internal erosion and piping is to determine, as reliably as is practical, the zoning of the embankment, and what materials were used to build the dam and their properties, and geology of the foundation and the properties of the materials in the foundation.

The authors' experience is that it is not sufficient to rely on design drawings and the specification and assume those who built the dam followed the specification. Too often it has been found that filters have been used with higher fines content or coarser than specified, particularly near the crest of the dam, as borrow areas or stockpiles or both became worked out.

There seemed to be a lack of awareness in the 1950s and 1960s of the need for good filters near the crest of the dam, so the quality control was sometimes relaxed. Some dams even had the filters omitted near the crest (even though they were shown on drawings). Such "sins and omissions" were not restricted to small organisations – it happened in large organisations with a good reputation as dam engineers and it often happened systemically.

The authors prefer to have available the following:

- Particle size distributions on samples taken from the dam during construction ("record testing").
- As-constructed drawings.
- Photographs of construction (illustrate whether, for example, filters were placed ahead of or after rockfill, or were placed in high lifts with subsequent risk of segregation).
- Test pits or drilling (e.g. using sonic drilling) into the dam, to log and sample the relevant zones, with associated particle size distributions.

9.3.2 Continuing and excessive erosion criteria

Foster (1999), Foster and Fell (1999a, 2001) used the test data from Sherard (1984a, b) and additional tests using no-erosion test equipment similar to that shown in Figure 9.7 (shown in Figure 9.17) to develop the concept of no erosion, some erosion, excessive erosion and continuing erosion shown in Figure 9.16.

The terms are defined as:

(i) *No erosion:* filter seals with practically no erosion of the base material.
(ii) *Some erosion:* filter seals after "some" erosion of the base material.
(iii) *Excessive erosion:* filter seals, but after "excessive" erosion of the base material.
(iv) *Continuing erosion:* the filter is too coarse to allow the eroded base materials to seal the filter.

The continuing erosion boundary was determined by carrying out additional filter tests, using a modified version of the NEF test, called Continuing Erosion Filter (CEF)

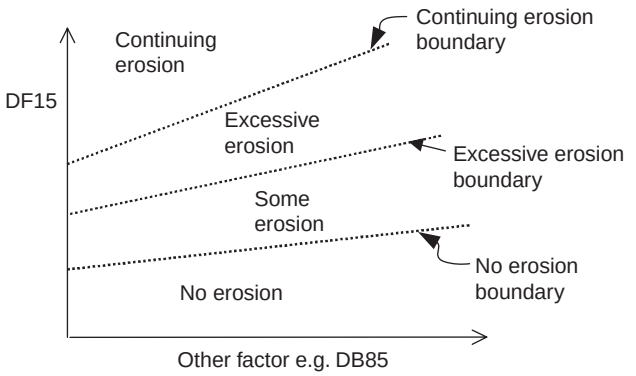


Figure 9.16 Filter erosion boundaries (Foster, 1999; Foster and Fell, 1999a, 2001).

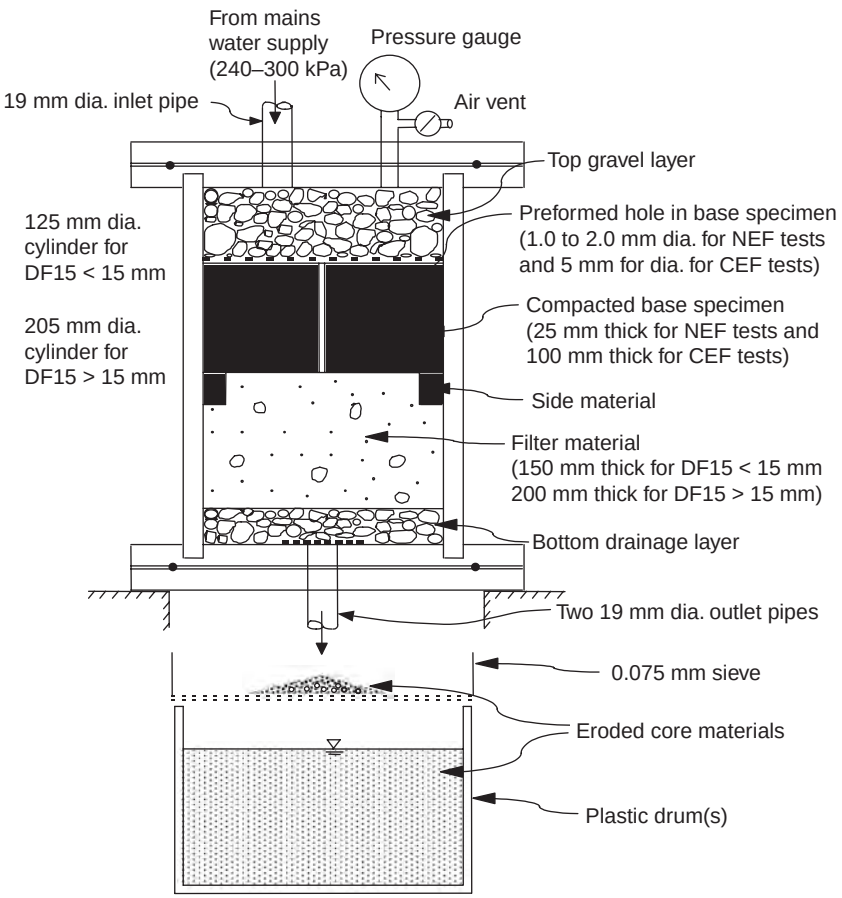


Figure 9.17 Continuing erosion filter test apparatus (Foster and Fell, 1999a, 2001).

tests, to determine the continuing erosion boundary for soils with $D_{85B} > 0.1$ mm. The test procedures of the CEF tests were essentially the same as those of the NEF test, as described by Sherard and Dunnigan (1989), but with the following modifications:

- Water passing through the filter during the tests was collected and the eroded materials dried and weighed to determine the loss of base soil required to seal the filter.
- Progressively coarser filters were used until the filter was not sealed.
- Thicker base specimens were used to allow for greater erosion losses.

The tests were carried out for sufficient time until it was evident the filter was sealed or it was judged that the filter was not going to seal no matter how much erosion of the base soil occurred. The filters were judged to have sealed when all of the following conditions were reached:

- Full mains pressure was maintained in the space above the base specimen as measured on the pressure gauge.
- Water passing through the filter was clear.
- The flow rate of water passing through the filter had decreased substantially from the initial flow and was relatively constant.

Details of the tests and the results of testing are given in Foster (1999), Foster and Fell (1999a, 2001).

The excessive erosion boundary was determined from consideration of the performance of dams which had resulted in damage to the dam in the form of sinkholes and large leakages, but none had resulted in failure (i.e. breaching) of the dam (Foster and Fell, 1999b).

Figure 9.18 presents the gradations of the filters of some of the dams with poor filter performance. The case histories generally involved internal erosion and piping of core materials into coarse or segregated downstream filters in zoned earthfill or central core earth and rockfill dams. The dams were generally constructed in the 1960s to 1970s which coincides with a period when there was a trend away from the use of uniformly graded multiple filters and towards the use of a single filter of substantial width and broad gradation (Ripley, 1994). The filter gradings shown in Figure 9.18 have wide gradings and low proportions of sand sizes which would tend to make them susceptible to segregation during construction and also potentially make them internally unstable.

There are also several reported cases of concentrated leaks that have developed through the cores of dams but which have evidently sealed due to the effectiveness of the downstream filter, as evidenced by observations of near hydrostatic piezometer levels in the downstream section of the core and "wet seams" in the core (Sherard, 1985a). Peck (1990) also describes several examples from the literature of dams, which have shown evidence that some form of filtering action has taken place at the core-filter interface.

Only two dams, Rowallan Dam and Whiteman's Dam, were found in the literature, which have experienced poor filter performance involving internal erosion and piping of fine-grained core materials with $D_{95B} < 2$ mm. In both cases, the finest core material

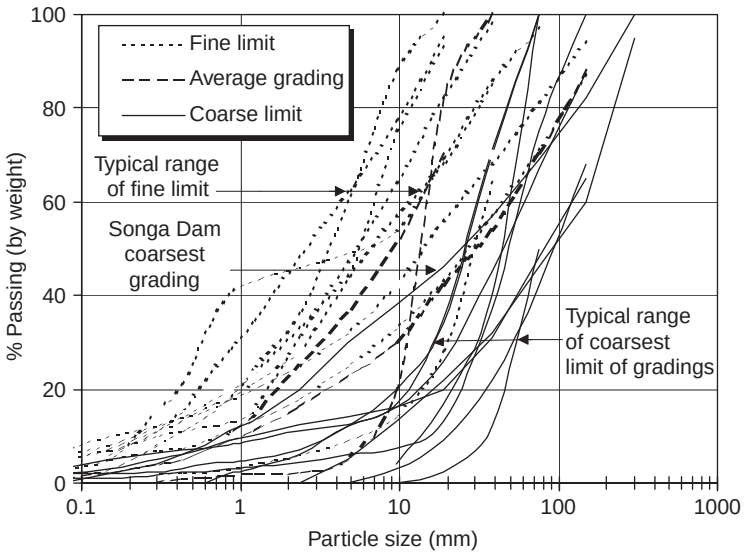


Figure 9.18 Gradings of filters which have experienced poor filter performance.

and coarsest filter combination fell into the continuing erosion category as defined by the laboratory tests, i.e. $D_{15F}/9 > D_{95B}$. At Rowallan Dam where very fine abutment contact earthfill was protected by the filter provided for the bulk of the earthfill, the filter with the coarsest grading has a filter opening size ($D_{15F}/9$) of $11/9 = 1.2$ mm. This is larger than the D_{95B} of the finest grading of contact clay soil of 0.9 mm. At Whiteman's Dam, core materials were eroded into the downstream gravel zone; the filter opening size of this coarsest gravel zone material was 1.0/9–0.1 mm and the D_{95B} of the finest core grading is 0.075 mm.

The other dams with poor filter performance generally have broadly graded core materials which fall into Soil Groups 2 and 4 (fines content 15–85%) and have $D_{95B} > 2$ mm, which places them in the soil types where a continuing erosion boundary could not be identified by the CEF tests. Figure 9.19 shows the range of D_{15F} of the filter plotted against the average percentage of fine-medium sand sizes (% 0.075 mm to 1.18 mm) of the core material for the dams which have had poor and good filter performance. The contours of equal erosion losses from the CEF tests and the no erosion boundary for Soil Group 2 soils ($D_{15F} = 0.7$ mm) are shown on the plot. The percentage fine-medium sand has been taken off the grading curves of the core materials after adjustment to a maximum particle size of 4.75 mm.

Dams with good filter performance generally have filters with an average $D_{15F} \leq 0.5$ mm, which is finer than the Sherard and Dunnigan (1989) design criterion for Soil Group 2 base soils ($D_{15F} \leq 0.7$ mm). The coarsest gradings are only slightly coarser than this (D_{15F} up to 1.5 mm).

Dams with poor filter performance have filters with an average $D_{15F} > 1.0$ mm and generally with D_{15F} greater than or equal to about the 0.25 g/cm^2 contour of erosion loss. Where a range of filter gradations is given, the coarsest grading is significantly

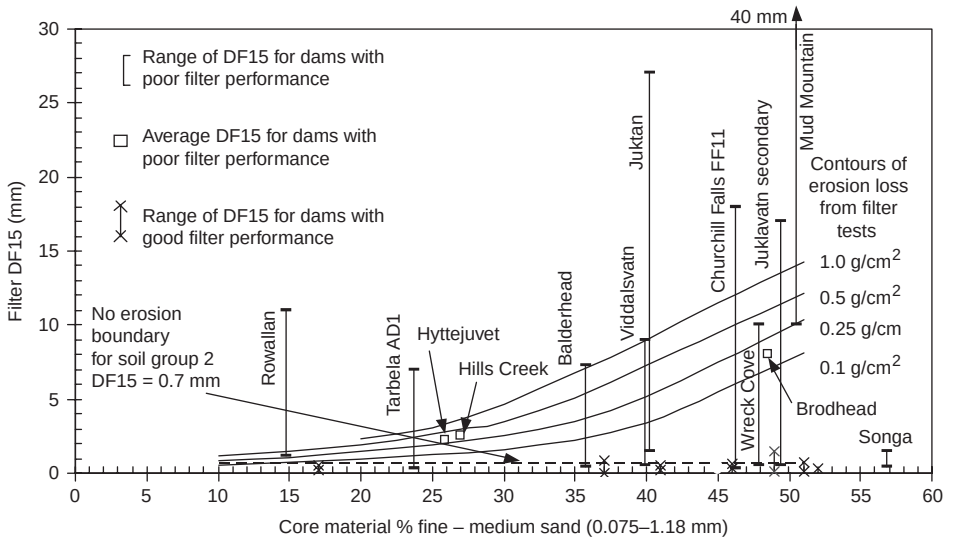


Figure 9.19 Comparison of erosion losses measured in filter tests to dams with poor and good filter performance.

coarser than the design criteria. Balderhead Dam has the finest coarse limit grading of the filter ($D_{15F} = 7$ mm) and this is 10 times coarser than the recommended design criteria for a non-erosion filter. The D_{15F} for the coarsest gradings is typically greater than or equal to about the 1.0 g/cm^2 contour.

One notable exception is Songa Dam, which has a range of D_{15F} of 0.4–1.5 mm. This is considerably lower than the other dams with poor filter performance. However, the gradings of the filter for Songa Dam (Figure 9.18) have a wide grading and low proportion of sand sizes which, as discussed later on, would have made the filters particularly susceptible to segregation during placement. Therefore it is likely that the actual gradings of the filter in this dam are probably locally much coarser than that shown.

Table 9.11 shows the criteria for the excessive and continuing erosion boundaries. The criteria for the excessive erosion boundary are selected from the case studies and the laboratory testing.

Based on case data and the laboratory tests they suggested the likely leakage behaviour would be:

- *Seals with No Erosion* – the filtering material stops erosion with no or very little erosion of the material it is protecting. The increase in leakage flows is so small that it is unlikely to be detectable.
- *Seals with Some Erosion* – the filtering materials initially allow erosion from the soil it is protecting, but it eventually seals up and stops erosion. Leakage flows due to piping can be up to 100 l/s, but are self healing.
- *Seals with Excessive Erosion* – the filter material allows erosion from the material it is protecting, and in the process permits large increases in leakage flow (up to

Table 9.11 Criteria for the excessive and continuing erosion boundaries (Foster, 1999; Foster and Fell, 1999a, 2001).

Base soil	*Proposed criteria for excessive erosion boundary*	*Proposed criteria for continuing erosion boundary*
Soils with $D_{95B} < 0.3$ mm	$D_{15F} > 9D_{95B}$	
Soils with $0.3 < D_{95B} < 2$ mm	$D_{15F} > 9D_{90B}$	For all soils:
Soils with $D_{95B} > 2$ mm and fines content > 35%	Average $D_{15F} > D_{15F}$ which gives an erosion loss of 0.25 g/cm^2 in the CEF test (0.25 g/cm^2 contour line in Figure 9.19) or Coarse limit $D_{15F} > D_{15F}$ which gives an erosion loss of 1.0 g/cm^2 in the CEF test (1.0 g/cm^2 contour line in Figure 9.19)	$D_{15F} > 9D_{95B}$
Soils with $D_{95B} > 2$ mm and fines content <15%	$D_{15F} > 9D_{85B}$	
Soils with $D_{95B} > 2$ mm and fines content 15–35%	$D_{15F} > 2.5\ D_{15F}$ design, where D_{15F} design is given by: D_{15F} design = $(35 - \text{pp\% } 0.075\text{ mm})(4D_{85B} - 0.7)/20 + 0.7$	

Note: Criteria are directly applicable to soils with DB95 up to 4.75 mm. For soils with coarser particles determine DB85 and DB95 using grading curves adjusted to give a maximum size of 4.75 mm.

1000 l/s), but the flows are self healing. The extent of erosion is sufficient to cause sinkholes on the crest and erosion tunnels through the core.

- *Continuing Erosion* – the filtering material is too coarse to stop erosion of the material it is protecting and continuing erosion is permitted. Unlimited erosion and leakage flows are likely.

Whether a dam can withstand such flows without breaching depends on the discharge capacity of the downstream zone and whether unravelling or slope instability may occur. It is also likely that, if a dam experiences a piping event which does eventually seal, it will experience another later, as the erosion process moves laterally.

The criteria listed in Table 9.11 should be used with caution and, for final decision making, should be supported by laboratory tests using the filter/transition and core materials from the dams. The results should be tempered with sound dam engineering judgement.

There are some broader implications to these studies in that it is apparent that, for most soils, there is a considerable margin between design no-erosion and continuing erosion criteria. This assists in explaining why, despite the statistical variability in the particle size distributions of the base soil and filters, (which might on first consideration imply that there was a significant potential for piping to occur), this is unlikely because the no-erosion and continuous erosion boundary criteria are not exceeded. Figure 9.20(a) shows this conceptually.

However it is important to recognise that for fine ground base soils, the separation between the design limit, no-erosion and continuing erosion boundaries is much

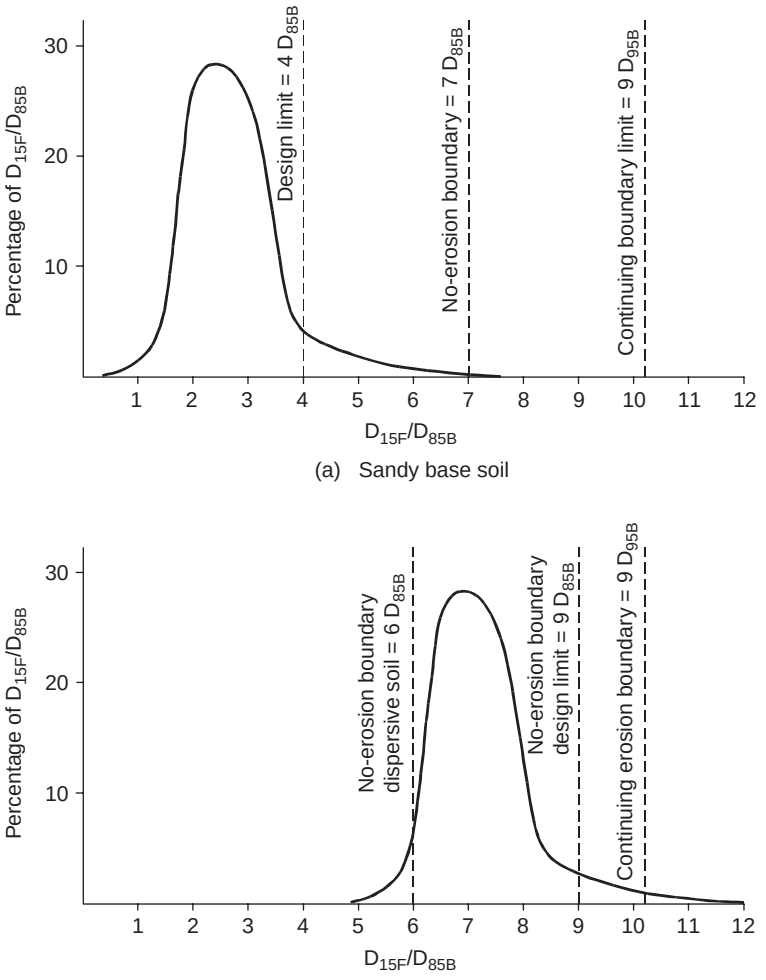

Percentage of D_{15F}/D_{85B}
30
20
10
Design limit = 4 D_{85B}
No-erosion boundary = 7 D_{85B}
Continuing boundary limit = 9 D_{95B}
1 2 3 4 5 6 7 8 9 10 11 12
D_{15F}/D_{85B}
(a) Sandy base soil
Percentage of D_{15F}/D_{85B}
30
20
10
No-erosion boundary dispersive soil = 6 D_{85B}
No-erosion boundary design limit = 9 D_{85B}
Continuing erosion boundary = 9 D_{95B}
1 2 3 4 5 6 7 8 9 10 11 12
D_{15F}/D_{85B}
(b) Fine clay and silt, little or no sand sized particles

Figure 9.20 Conceptual distribution of D_{15F}/D_{85B} for (a) clean sandy base soil, (b) base soil fine clay and silt with little or no sand sized particles, with design, no erosion and continuing erosion criteria.

less – Figure 9.20(b) and Tables 9.2 and 9.11. Hence if the filter design is close to the limiting D_{15F}/D_{85B}, very close control on construction will be necessary.

It is important to recognize the limitation of the laboratory test apparatus used to assess the performance of a filter. If the filter is a non-erosion filter or perhaps a filter with a slightly coarser grading, the ultimate seal may only affect a very small area of the soil/filter interface in the test. On the other hand, for a very coarse filter, one must be cautious when deciding if this test indicates "ultimate" sealing or whether it shows significant localized weakening of the soil with the high likelihood of the leak moving sideways. Care must also be taken in the laboratory test to prevent erosion up the sides of the sample.

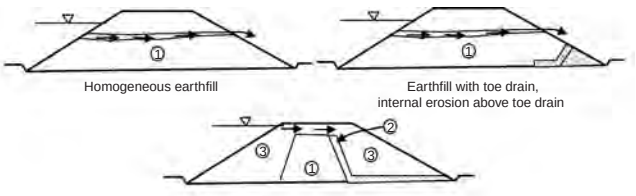
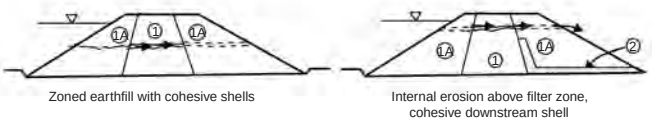
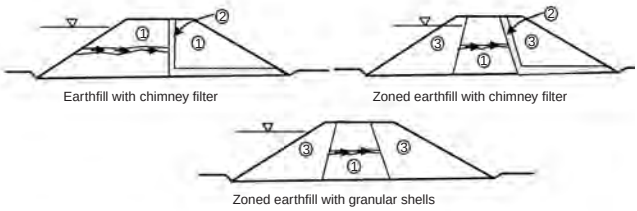
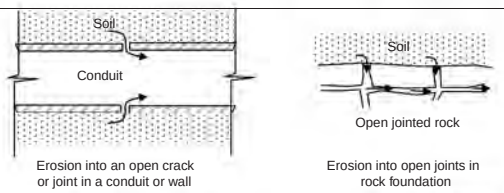
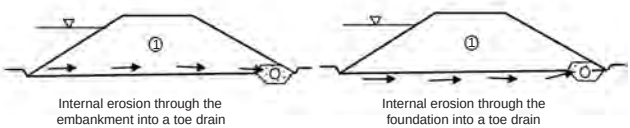
Scenarios	Examples
Scenario 1: Homogeneous zoning with no fully intercepting filter	
Scenario 2: Downstream shoulder of fine grained cohesive material which is capable of holding a crack/pipe.	
Scenario 3: Filter/transition zone is present downstream of the core or a downstream shoulder zone which is not capable of holding a crack/pipe.	
Scenario 4: Piping into an open defect, joint or crack.	
Scenario 5: Erosion into a toe drain	

Figure 9.21 Scenarios for continuation of internal erosion in an embankment (Fell *et al*., 2008).

9.3.3 Discussion of continuation scenarios in existing dams

9.3.3.1 *Internal erosion in the embankment, from the embankment into the foundation or into openings in conduits passing through the embankment*

There are a number of scenarios which may be applicable depending on the zoning of the dam and the failure mode under consideration. Example sketches are shown in Figure 9.21.

- Scenario 1: Homogeneous zoning with no fully intercepting filter or transition.

- Scenario 2: Downstream shoulder of fine grained cohesive material which is capable of holding a crack/pipe. Examples of soils which are capable of holding a crack or pipe are:
  - well compacted shoulder (shell), containing > 5% plastic fines
  - poorly compacted shoulder (shell), containing > 15% plastic fines
  - well compacted shoulder, containing > 30% non plastic fines
  - poorly compacted shoulder, >30% non plastic
- Scenario 3: Filter/transition zone is present downstream of the core or a downstream shoulder zone which is not capable of holding a crack/pipe. This includes earthfill dams with a chimney filter.
- Scenario 4: Erosion into a crack or open joint (e.g. open joint or crack in a conduit or adjoining concrete structure).
- Scenario 5: Erosion into a toe drain.

The manner in which the likelihood of continuation of erosion for these scenarios is considered differently:

Scenario 1. There is no potential for filtering. Continuing erosion is certain.

Scenario 2. The issue for this scenario is whether the crack/high permeability feature that is present through the core is continuous through the downstream shoulder, or if not, whether it can find an exit. This depends on the following factors:

- The mechanism causing the concentrated leak, in particular whether it also causes cracking in the shoulder.
- The material characteristics and width of the downstream shoulder zone.

Scenario 3. Assess the likelihood of the filter or transition zone being effective using the Foster (1999), Foster and Fell (1999a, 2001) filter design criteria.

Scenario 4. For erosion to continue through an open defect, the defect needs to be sufficiently open to allow the soil surrounding the defect to pass through it. The suggested procedure for assessing this is given in Section 9.3.5.

Scenario 5. This scenario is applicable if the failure path under consideration involves a seepage path that exits into a toe drain which could lead to continuing erosion of the embankment or foundation materials. The assessment considers the design and construction details of the toe drain, whether filter criteria are met as described in Section 9.2.7.1, and the observed condition of the toe drain (from video or external inspections).

9.3.3.2 *Internal erosion in the foundation*

The likelihood of continuation of erosion should be estimated first by assessing the likelihood that the exit will be a filtered or unfiltered exit.

(a) Given the exit is unfiltered continuing erosion is certain.
(b) Given the exit is filtered, estimate the likelihood of continuation using the method described in Section 9.3.4.

In risk based assessments the probability of continuation will be the product of the probability of a filtered exit and the probability assessed considering the filters.

It needs to be recognized that low permeability strata beneath horizontal drains may prevent them working effectively, Figure 9.22 shows an alluvial foundation where the lower permeability strata (A and E) will prevent the seepage in the most permeable sand and gravel strata (B, and D) from flowing into a filtered exit in the horizontal drain.

9.3.3.3 *Internal erosion of the embankment at or into the foundation*

(a) *Erosion into open joints in the foundation*. The likelihood of continuation depends on the joint openings being sufficiently open and continuous to allow erosion of the core materials.
(b) *Erosion into coarse grained soil foundation*. The likelihood of continuation depends on the coarse grained soils being sufficiently coarse and continuous to allow erosion of the core materials

These assessments are often difficult to make because the amount of information on the foundations is often limited. If there are soils overlying the exits to the open joints or the coarse grained soils these may act as filters to the erosion.

The assessment of filtering should be carried out as detailed in Sections 9.3.4 and 9.3.5.

9.3.4 Assessment of the likelihood of continuation where a filter/transition zone does not satisfy no-erosion filter criteria

9.3.4.1 *General principles*

The following steps should be followed when assessing the likelihood of continuation:

1. Gather the available information on particle sizes of the core and the filter/transition materials. This may include data from construction control testing, testing on samples taken from the dam and its foundation, data from investigations of borrow areas used for core and filter materials.
2. Plot the available information so the range of particle size distributions for all materials can be assessed.
3. Assess if the filter/transition materials are internally unstable and, if so, adjust the particle size distributions to account for this using the procedure outlined in Section 8.5.3.
4. Assess if the filter/transition materials are likely to be subject to segregation taking account of the specified and/or measured particle size distribution, the method of placement, and the width of the filter/transition zone as described in Sections 9.5.1 and 9.5.2.
5. Use modern filter design criteria, e.g. the method described in Section 9.2.4 or Sherard and Dunnigan (1989), to assess whether the filter/transition will prevent continuation. If the filter or transition does not satisfy no-erosion criteria, use the Foster and Fell (1999a, 2001) method to assess whether the filter/transition satisfies some, excessive or continuing erosion criteria. In most cases there will be a range of gradations of the core and filter/transition so some chance of the filter falling into each category as described in the example below.
6. Check for a "blow-out" condition. As described in Section 9.2.8.5.

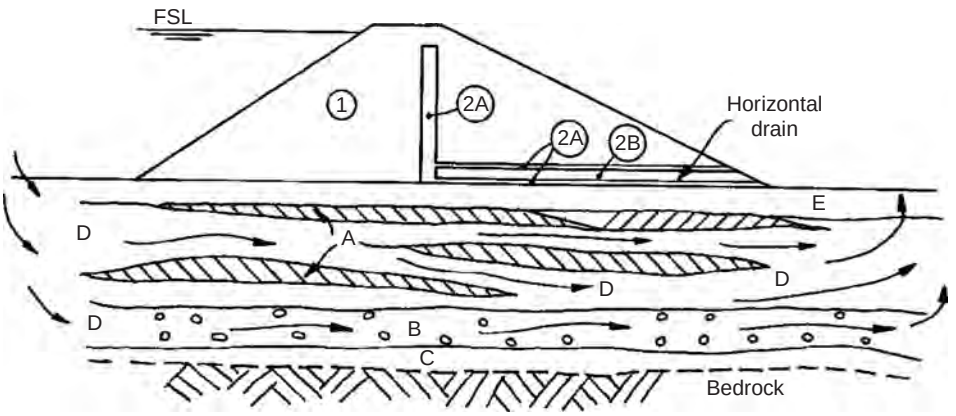


Figure 9.22 Schematic example of an embankment founded on a stratified alluvial soil where much of the seepage flow will be to an unfiltered exit.

9.3.4.2 *Details of how to apply the Foster and Fell (1999a, 2001) method for assessing the likelihood of continuation of erosion for filters and transitions which do not meet modern filter design criteria*

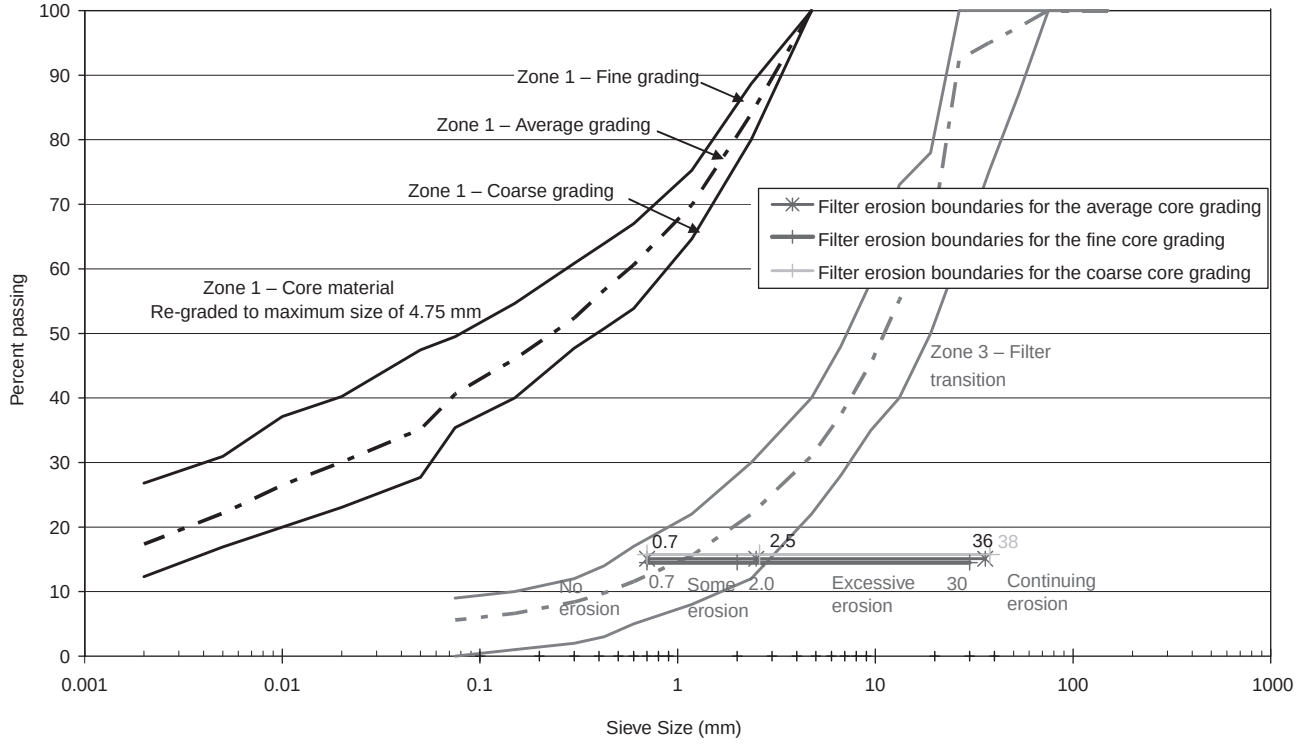
The following steps should be followed:

1. Evaluate the D_{15F} values for the no, excessive and continuing erosion boundaries using Table 9.11. Do this for the finest base soil grading, the average base soil grading and the coarsest base soil grading. Plot the D_{15F} values for these boundaries on the grading curve limits of the filter/transition material (see Figure 9.23 for an example). Use the adjusted grading curves for the filter/transition zone if required to do so by the preceding steps 3 or 4.
2. Estimate the proportion of the filter/transition gradings that fall into each of the no-erosion, some-erosion, excessive and continuing erosion for each representative base soil grading. Estimate these proportions based on the plot of filter/transition grading curves versus Filter Erosion Boundaries. More detailed guidance on how to do this is given in Appendix A.

It should be noted that as for the example in Figure 9.23, in dams the filters can be significantly coarser than those required for no-erosion filters, and still eventually seal with only some erosion of the protected soil, and relatively small leakage flows.

9.3.5 Assessment of the likelihood of continuation for internal erosion into an open defect, joint or crack in the foundation, in a wall or conduit

There are no commonly adopted criteria for assessing the likelihood of continuation for this scenario. Fell *et al.* (2008) suggest use of the criteria in Table 9.12 which have been determined by assuming that the Foster and Fell (1999a, 2001), continuing erosion criteria apply to erosion into a open defect, joint or crack and that crack width



Assessment of zone 1 core against no erosion, excessive erosion and continuing erosion criteria

Core gradation	Base soil sizes (mm)				No erosion	Excessive erosion	Continuing erosion
	DB85 (mm)	DB95 (mm)	% passing 0.075 mm	% fine-medium sand (0.075–1.18 mm)	DF15 (mm)	DF15 (mm)	DF15 (mm)
Fine grading	1.9	3.3	50	25	0.7	2	30
Average	2.4	4	41	29	0.7	2.5	36
Coarse Grading	2.5	4.2	35	30	0.7	2.6	38

Figure 9.23 Example of plot showing filter/transition gradings compared to filter erosion boundaries (Fell *et al.*, 2008).

Table 9.12 Continuing Erosion criteria for erosion into an open defect (Fell *et al.*, 2008).

Erosion Condition	*Comparison of Soil Gradation to Joint/Defect Opening Size (JOS)*
Continuing erosion (CE)	$JOS_{CE} = D_{95}$ *surrounding soil*

Notes: • JOS_{CE} = Joint/defect opening size that would allow continuing erosion of the surrounding soil.
• D95 should be based on the average soil grading after re-grading on 4.75 mm particle size.

is equivalent to the filter opening size of the voids between the particles in a filter. The filter opening size has been shown by Sherard *et al.* (1984a) to be $D_{15}/9$. This is discussed further in Section 9.6.2.2.

9.4 SPECIFICATION OF PARTICLE SIZE AND DURABILITY OF FILTERS

9.4.1 Particle size distribution

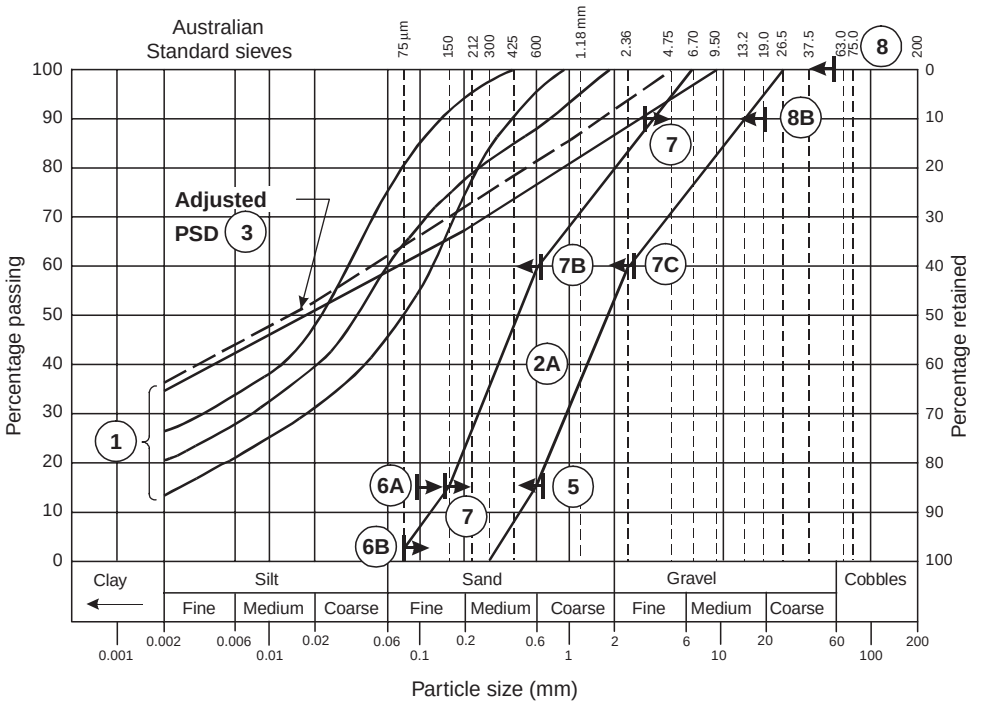
The design rules outlined in Section 9.2.4 do not in themselves entirely allow selection of the particle size distribution of filters. The emphasis in those rules is on "spot" limitations within the grading curve. For filters to be manufactured and placed in a dam the full particle size distribution limits have to be designed as for example in Figure 9.24.

Some factors which should be taken into account are:

- The wider the particle size grading allowed and the more compatible this is with the available sand and gravel, the lower the cost of manufacture. If natural sand and gravel deposits are available it may sometimes, but not often, be possible to match the filter grading requirements to the grading of those deposits. More commonly it may be possible to use a gradation which matches a product the quarry is already producing, e.g. for concrete aggregates.
- Breakdown of particles (particularly coarser particles) occurs during placement and compaction. Specifications should apply to the filter after placement in the dam. This means that the contractor may have to manufacture the filters coarser than the specifications. In particular fines contents may need to be lower for the filters delivered to the site to allow for some increase during placement.
- On most large new dam projects concrete aggregates and filters are produced in the same crushing and screening plant, so sieve sizes on which specifications are based should be the same for both.

An example of design of Zone 2A and 2B filters is given in Figures 9.24 and 9.25. The "steps" are those detailed in Section 9.2.4.

Most experienced dam designers, including the authors, design the Zone 2A filter for the finest soil to be used in the dam, or which has been identified by investigations of an existing dam. It is not, in the view of the authors, defensible to design on an



Notes: Step 1 – Plot particle size distributions (PSD); Step 3 – Adjust PSD for gravelly soils; Step 4 – Select base soil category for finest soil – Type 2; Step 5 – Determine D_{15Fmax} from Table 9.4 = 0.7 mm; Step 6 – Determine permeability criterion; D_{15Fmin} = 0.1 mm, 6(A), less than 2% passing 0.075 mm (6B); Step 7 – D_{XFmax}/D_{XFmin} less than 5 e.g. D_{15Fmax} = 0.7 mm; D_{15Fmin} = 0.14 mm (7); D_{60}/D_{10} less than 6, say D_{10Fmin} = 0.12; D_{60Fmin} = 0.72 (7B); D_{10Fmax} = 0.45, D_{60Fmax} less than $5D_{10Fmax}$ = 2.7 mm (7C); Step 8A – D100F less than 50 mm or 75 mm, say 50 mm (8); Step 8B – D_{90Fmax} = 20 mm (Table 9.7).

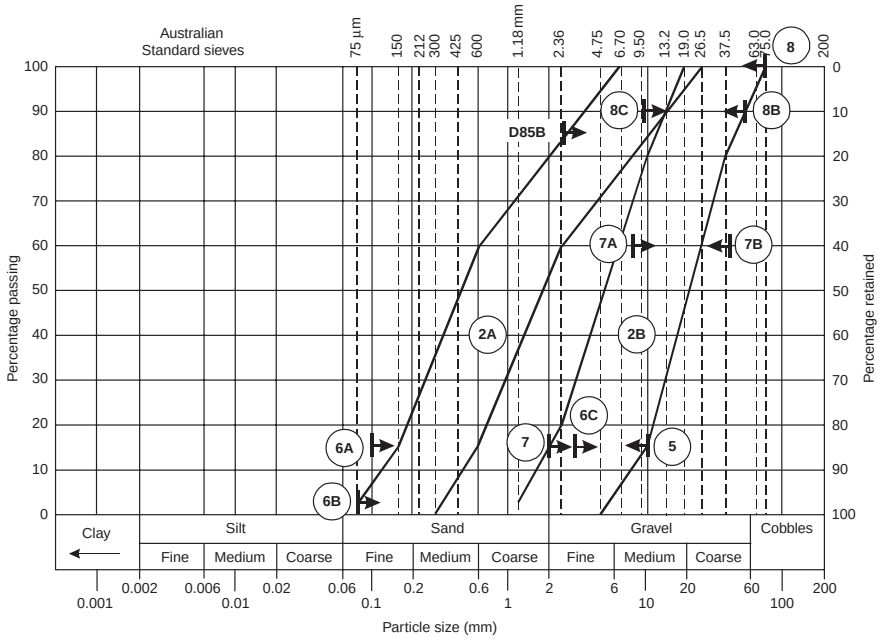
Figure 9.24 Example of design of a Zone 2A filter.

average grading – the filters are required to stop all soils in the dam core or foundation eroding, not just half of them!

If one can be confident that finer soils in the proposed borrow area for a new dam will be mixed with coarser soils by the loading, dumping and spreading of the soil, the Zone 2A filter may be designed for other than the finest soil in the borrow area. However this is difficult to control and few natural deposits will have sufficiently low fines content to be used without washing to remove fines.

In the example in Figures 9.24 the D_{85B} is less than 0.075 mm, and must be obtained from hydrometer particle size analysis. This should be done using the standard approach, in which dispersant is added to the soil as part of the test procedure.

For Zone 2B, a maximum particle size of 75 mm has been adopted. Larger size particles may be used in Zone 2B filters but at the risk of segregation in stock piles and during placement and the authors would not recommend this be done.



Notes: Step 1 – Particle size distribution for zone 2A becomes the 'Base Soil' – the fine side boundary is critical. D_{85B} = 2.5 mm. Go to Step 4. Step 4 – Soil is type 4 (<15% passing 0.075 mm). Step 5 – Determine D_{15Fmax} from Table 9.4. D_{15Fmax} less than $4D_{85B}$ = less than 4×2.5 mm = 10 mm (5). Step 6 – Determine permeability criterion, D_{15Fmin} = 0.1 mm, (6A), 2% passing 0.075 mm (6B), $D_{15Fmin} = 4D_{15B} = 4 \times 0.7$ mm or = 2.8 mm. However for average $D_{15} = 4 \times 0.3$ mm = 1.2 mm accept this (6C). Step 7 – D_{XFmax}/D_{XFmin} less than 5 e.g. D_{15Fmax} = 10 mm, therefore D_{15Fmin} = 2 mm (7A), D_{60}/D_{10} less than 6, so say D_{10Fmax} = 6.7 mm, D_{60Fmax} = 40 mm (7B); Step 8 – D_{10F} = 5 mm, so max D_{90F} = 50 mm from Table 9.7 (8).

Figure 9.25 Example of design of a Zone 2B filter.

9.4.2 Durability

Filters are required to be constructed of sand and gravel which is sufficiently durable not to break down excessively during the mechanical action of placement in the dam under the chemical action of seepage water or under wetting and drying within the dam.

9.4.2.1 *Standard tests for durability and particle shape*

Table 9.13 shows a typical specification for durability and particle shape for Zone 2A and Zone 2B filters. These are not greatly different to what is required for concrete aggregates.

These specifications can be used as a guide to selecting reasonable limits. The following comments are offered:

(a) The Los Angeles abrasion and wet strength, wet/dry strength variation tests largely assess the susceptibility to breakdown under the mechanical action of placement and rolling in the dam. It can be seen that dam specifications are generally less stringent than AS2758.1. If materials test marginally within the

Table 9.13 Typical durability and other requirements for Zone 2A (fine) and Zone 2B (coarse) filters.

Test Type	*Australian Test Standard*	*ASTM Test Standard*	*Acceptability Criteria*
Abrasion Resistance	AS 1141.23	ASTM C131	Zone 2B filter: 30% maximum loss of weight at 1000 revolutions
Sodium Sulphate Soundness	AS 1141.24	ASTM C88	All filters (Zone 2A, 2B): 10% maximum weighted average loss by weight after five cycles
Specific Gravity (Particle Density)	AS 1141.5 (<4.75 mm) AS 1141.6.1 (>4.75 mm)	ASTM C127	All filters (Zone 2A, 2B): 2.60 t/m^3 minimum
Water Absorption	AS 1141.5 (<4.75 mm) AS 1141.6.1 (>4.75 mm)	ASTM C127	All filters (Zone 2A, 2B): 2.0% maximum
Wet Strength Wet/Dry Strength Variation	AS 1141.22	N/A	Zone 2B and Zone 2A filter if Zone 2A has a significant % of gravel: Wet strength: minimum 80 kN Wet/Dry Variation: 35% maximum
Atterberg Limits and Plasticity Index	AS 1289.3.1.2, .3.2.1 and .3.3.1	D4318	All filters (Zone 2A, 2B): Liquid limit not obtainable or Plasticity Index = 0%.
Misshapen particles	AS 1141.14	N/A	Zone 2B only: Total percentage of misshapen particles not to exceed 10% for 3:1 ratio
Flakiness Index	AS 1141.15	D4791	Zone 2B only: Flakiness index not to exceed 35%

specification it would be advisable to carry out field compaction trials of the filters, to observe directly the degree of breakdown under rolling and the size of the "broken down" product.

(b) The Sodium Sulphate soundness test assesses the susceptibility to breakdown under wetting and drying, with the added involvement of sodium sulphate in the solution used for soaking. The sodium sulphate penetrates fine cracks in the aggregates and expands on crystallization in the drying phase, breaking apart the aggregate particles.

The test is particularly severe for most dam applications where high salt contents are not present. This is reflected in the dam specifications in Table 9.13, where the requirements are less severe than for coarse concrete aggregates (similar for fine aggregates).

If materials are testing as marginal for this test, it is important to inspect the product of breakdown. In some rocks the product will be a fine silt and sand, which is likely to affect filter permeability markedly. In others it is simply the breakup of coarse

particles into two or three smaller particles, which does not greatly affect permeability or filter capability.

It is quite common to have different performance in these tests depending on the size fraction being tested. This is particularly the case for naturally occurring aggregates from a river which drains a mixed geological environment where the mineralogy of the individual particles may vary substantially.

The authors' opinion is that for important tailings dams which are storing water with a high acidity or high salts content, it may be necessary to impose more severe requirements on sulphate soundness than normally accepted for water dams. It may also be necessary to test with the more severe magnesium sulphate if the water in the tailings dam has high magnesium sulphate content.

9.4.2.2 *Possible effects if carbonate rocks are used as filter materials*

Rocks or natural aggregates containing significant proportions of carbonate minerals (e.g. limestone, dolomite, marble, gypsum, or calcite) may perform satisfactorily in the tests in Section 9.4.2.1, but could be unsuitable for use in filter zones, because any of the following effects may be possible, during the lifetime of the dam.

(a) Change of grading, due to dissolution.
(b) Partial dissolution and re-cementation.
(c) Interlocking of grains due to pressure-solution.

Of these (b) or (c) are more likely to be an issue and could cause a filter zone to become cohesive and be capable of sustaining an open crack or erosion tunnel. All 3 possible effects are discussed in more detail in Sections 3.7.7 and 2.9.4.

The authors consider that carbonate materials should not be adopted for use in a filter zone, particularly fine filters, until the risk of malfunction due to any of the above, during the lifetime of the dam, has been assessed by experienced geochemical specialist or team.

9.4.2.3 *Effects if rocks containing sulphide minerals are used as filter materials*

Rocks containing small amounts (1% or less) of sulphide minerals may also perform satisfactorily in the tests in Section 9.4.2.1, but in service, the sulphides will oxidise and produce sulphuric acid and sulphate minerals. Effects can include clogging or cementation of the filter, by sulphate minerals (or iron hydroxides?) and acid drainage problems (see Sections 2.9.4 and 3.7.8). The effects may also include weakening of the filter rock and acid attack on other embankment materials.

The authors recommend that rocks containing sulphide minerals should be avoided, whenever possible, when selecting filter zone materials.

9.4.2.4 *Other investigations for filter materials*

In the light of all of the above, the authors recommend that as well as the laboratory tests discussed in Section 9.4.2.1, the following should always be undertaken

when investigating sources of filter aggregates, or when investigating existing filters:

(i) The mineral composition of the particles should be determined regardless of whether it is quarried and crushed rock, or alluvial sand/gravel.
(ii) If material is found to contain even small amounts (e.g. 1% to 5%) of carbonates, gypsum, or much less than 1% of sulphide minerals, then it should be subjected to further laboratory and field studies to assess its likely performance in the proposed filter zone.

In the field the studies are primarily of the observational type. For all types of materials a search should be made for existing (preferably very old) deposits of the materials, e.g. old fills, pavements or natural scree, or for filters in an existing dam, trenches or pits dug into the filters. In the case of alluvial materials being considered for use, the actual deposit in question is also studied.

The deposits are carefully exposed by trenching or pitting. Evidence of any cohesive behaviour is looked for during excavation and the exposed faces and spoil are examined for any particles which have become cemented together. Evidence of weathering or solution of particles is also looked for.

When judging the suitability or otherwise of a material, from the results of such observational tests, differences between the environments (particularly moisture) in the old deposit and the proposed filter zone must be taken into account.

(iii) Volcanic ash materials should also be checked carefully for long term chemical stability and for any tendency to develop cohesion when compacted. As well as the laboratory and field tests described in (ii) above, field compaction trials are advisable for sand-sized volcanic ash materials.

9.4.3 Contractual difficulties associated with gradation and durability of filters

It is common to have contractual difficulties relating to the gradation and durability of filters. These occur at the supplier and contractor level. The following details some of these.

9.4.3.1 *Fines content*

Almost invariably suppliers and contractors will provide samples of proposed filter materials for approval which do not satisfy the requirement for limiting fines or have trouble during construction. This is due to a number of causes:

(a) They may not recognize the importance of these requirements and have not understood that they will be enforced.
(b) It often costs more to process the filters to reduce the fines content to specified level, and may require additional steps in the process plant which were not allowed for in contract bid prices.
(c) They may not have allowed for potential breakdown of the filters on compaction.
(d) Dry sieving may have been used in the supplier's tests when wet sieving should be used. As a result fines contents were under-estimated.

All that can be done is to spell out clearly in specifications the requirements and advise the tenderers that the specification will be enforced.

9.4.3.2 *Use of crushed rock for fine filters*

Many suppliers and contractors will attempt to manufacture fine filters by crushing rock. This has the advantage that one source and plant can supply coarse and fine filters. The problems with this are:

(a) Almost invariably the crushing process results in a deficiency in coarse sand size material and a surplus of fines. The first reaction from the contactor is to want to alter the specification. When the specification is enforced there is often a lot of wastage which the contractor claims was unexpected, so claims for the additional costs result.
(b) The crusher run product is often gap graded so the finer fraction may be eroded by seepage flows because the filter is internally unstable and so potentially outside the coarse limit
(c) This is particularly a problem when rock with fine grain crystals is used, such as basalt, dolerite, and some greywacke.

Part of the answer to this is to warn tenderers about the problems in the contract documents. The other important thing to do is to try to ensure the supplier and/or contractor has a skilled crushing plant team and the correct equipment.

9.5 DIMENSIONS, PLACEMENT AND COMPACTION OF FILTERS

9.5.1 Dimensions and method of placement of filters

9.5.1.1 *Some general principles*

The following factors should be taken into account when selecting the dimensions of zones of filters.

(a) For filters upstream and downstream of the dam core:

- The theoretical width to achieve filtering action (see below).
- The discharge capacity required along the filter zone e.g. for chimney drains (see Section 10.7).
- The particle size distribution of the filter and the potential for segregation. Wider filter zones should be used if segregation is likely and narrower zones may be used for more uniformly graded materials.
- Potential displacement under major earthquake – filters need to be wide enough to remain effective even if displacements occur in an earthquake.
- The size of the dam – there is a tendency to use minimum width of 2.5 m or 3 m for very large (>100 m to 150 m high) dams, to allow for differential movement during construction and simply for conservatism.
- The method of placement, as detailed below.

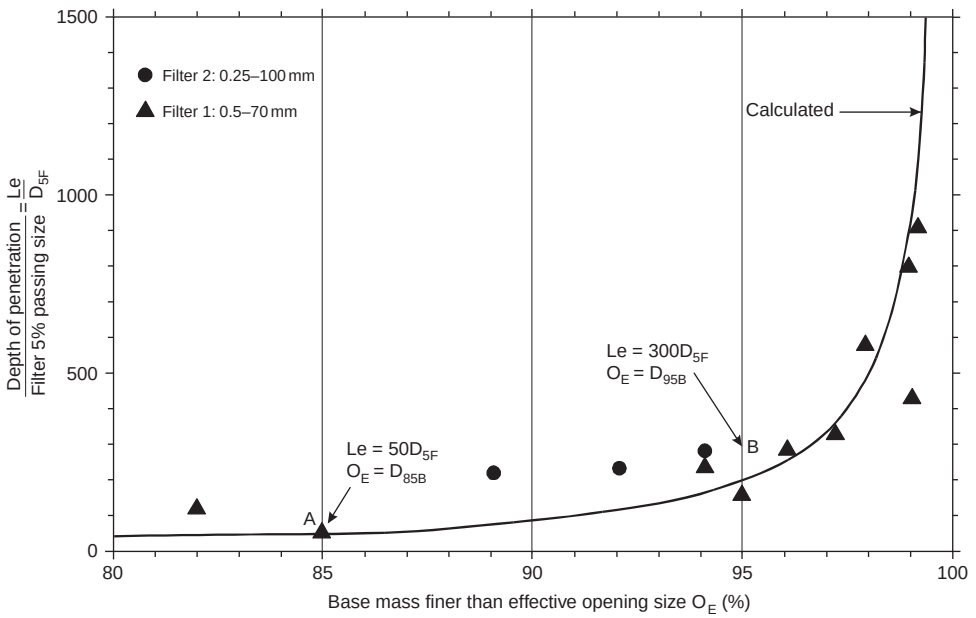


Figure 9.26 Depth of penetration of base soil into filter material (Le) versus effective opening size (O_E) (adapted from Witt, 1993).

(b) For horizontal, or near horizontal filters, e.g. in horizontal drains, on the surface of erodible foundations:

- The theoretical thickness to achieve filtering action (see below).
- The discharge capacity required along the filter zone, e.g. for a horizontal drain (see Section 10.7).
- The particle size distribution of the filter, the potential for segregation and maximum particle size. As a guide, the horizontal filter layer thickness should be not less than 20 times the maximum particle size, to ensure that, given good segregation control, there are very unlikely to be continuous coarse zones through the filter layer.
- The method of placement, as detailed below.

The theoretical minimum width or thickness for filters designed according to no-erosion criteria is very small and does not control the dimensions of filters. Witt (1986, 1993) carried out some calculations and experiments which demonstrated that the depth of penetration of the base soil into the filter is small, even if the filter is somewhat coarser than required by the design criteria described above.

Figure 9.26 shows the depth of penetration versus effective opening size O_E of the filter. The graph shows:

At A, $O_E = D_{85B}$, Depth Penetration $= 50D_{5F}$
and at B, $O_E = D_{95B}$, Depth Penetration $= 300D_{5F}$

Sherard and Dunnigan (1985, 1989) design criteria range from $D_{15F} \leq 4D_{85B}$ for Group 3 soils, to $D_{15F} \leq 9D_{85B}$ for Group 1 soils, so the depth of penetration will be less than $50D_{5F}$ for filters designed according to Sherard and Dunnigan criteria. Commonly, Zone 2A filters would have a D_{5F} less than 0.5 mm, so the depth of penetration would be less than 25 mm.

Larger depths of penetrations, and hence larger minimum thicknesses would be calculated using Witt (1986, 1993) approach for Zone 2A filters penetrating Zone 2B, but it is not clear if his work would apply to this case.

For assessing the filters in existing dams, the degree of compliance with these requirements would be taken into account.

9.5.1.2 *Placement methods*

Given that the width needed to achieve a proper filter against the protected soil is small, the width of filter zones is usually determined by construction requirements.

The selection of the economic dam zoning should account for filter placement requirements and the cost of manufacture of the filter materials. In most dam projects the cost of manufacture of filters is high and there is a need to keep widths to a practical minimum. The possible exception is the rare occasion when naturally occurring sand and gravel deposits satisfy filter design requirements and are therefore relatively inexpensive.

The following is offered as a guide to practical minimum widths:

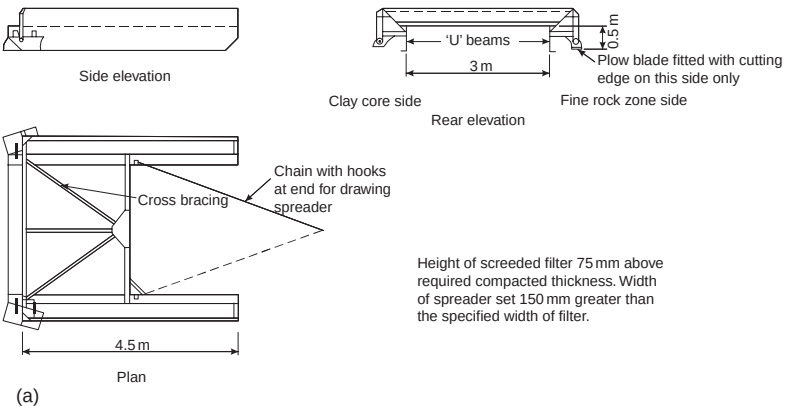
(a) Filters upstream or downstream of an earth core, when constructed by end-dumping off a truck should be at least 2.5 m and preferably 3 m wide.

(b) If a spreader box such as that shown in Figure 9.27 is used, a minimum width of 1.5 m is practicable. The filter material is dumped off the truck into the spreader box, which spreads the filter out of its base as it is pulled along by a small bulldozer. The authors are aware of one project where Zone 2A and 2B filters were placed concurrently using a double spreader box; in the odd case, three filters have been placed at the same time with a triple celled box.

(c) If filter materials are very scarce or high cost, formwork can be used to contain bands of filters as narrow as one metre. Sherard *et al.* (1963) show an example of such placement. This is very unusual and would only be contemplated in exceptional circumstances.

(d) For earthfill dams with a vertical chimney, a relatively narrow filter (as narrow as 0.75 m or 1.0 m) can be constructed by placing the earthfill for up to 2 m over the filter layer, and then excavating through the earthfill with a backhoe or excavator to expose the filter, as shown in Figure 9.28. Careful cleanup of the surface of the exposed filter is necessary and the filter is compacted with small vibrating sleds or other compaction equipment. The depth "h" is best limited to say 1.5 m to reduce the risk of collapsing of the trench and allow access of workers into the trench. Contamination of the filter during placement can be reduced by spreading it from a movable steel plate placed adjacent to the trench (Charlton and Crane, 1980). This also reduces the risk of collapse of the trench under the surcharge load of the filter material.

(e) For smaller dams, an inclined chimney drain can be constructed by dumping the filter on the trimmed downstream slope of the earthfill core as shown in


Side elevation
'U' beams
3 m
0.5 m
Plow blade fitted with cutting edge on this side only
Clay core side
Fine rock zone side
Rear elevation
Chain with hooks at end for drawing spreader
Cross bracing
Height of screeded filter 75 mm above required compacted thickness. Width of spreader set 150 mm greater than the specified width of filter.
4.5 m
Plan

(a)



(b)



(c)

Figure 9.27 (a) Typical spreader box and screed (adapted from Charlton and Crane, 1980), (b) Spreader box from the rear and (c) Double spreader box (FEMA, 2011).

Figure 9.29. The filter can be compacted by rolling up the slope or by running rubber tyred equipment up against the slope or by vibrating plate compactor. The downstream earthfill (or rockfill) is then placed in layers adjacent to the filter. In this way thin layers of filter (say as thin as 0.3 m normal to the slope,

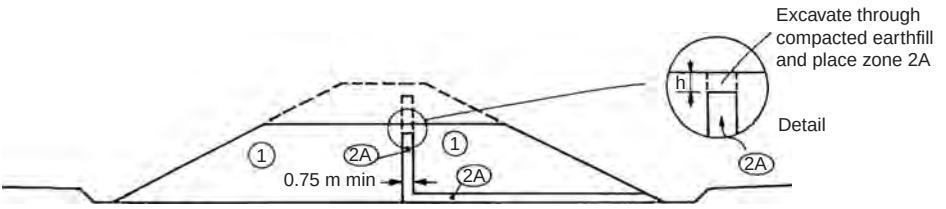


Figure 9.28 Construction of vertical chimney drain by excavation through earthfill.

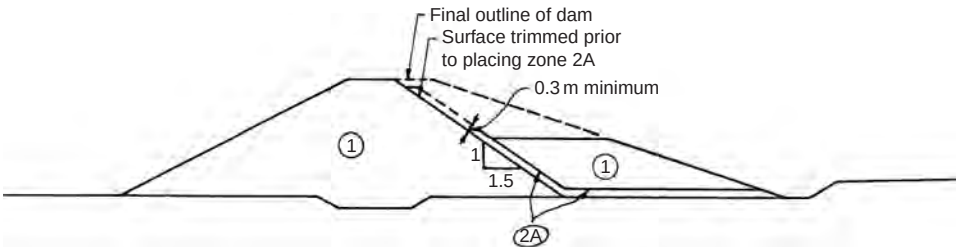


Figure 9.29 Construction of inclined chimney drain by placement on downstream slope of earthfill.

0.5 m horizontal for Zone 2A, 0.45 m normal to the slope, 750 mm horizontal for Zone 2B) can be placed. In practice it is difficult to place the filters so narrow and without segregation of the Zone 2B so if narrow widths are used a very high standard of construction supervision is essential.

Horizontal filters can be placed in layers as thin as 150 mm or as thick as about 500 mm after compaction (the upper limit being determined by compaction requirements). However, for thick layers of filters (e.g. 400 mm), it is considered preferable to place 2×200 mm layers rather than 1×400 mm layer since, if segregation occurs during placement, it is less likely that coarse zones will coincide with each other in the two layers.

9.5.2 Sequence of placement of filters and control of placement width and thickness

Filters generally should be placed ahead of the adjacent earthfill or rockfill zones as shown in Figure 9.30.

This is desirable because it allows good control of the width of the filter zone compared to the specified width and reduces the risk of contamination of the filter zone with materials from the adjacent zones and from water eroding soil fines off adjacent areas. Figure 9.31 shows the alternative which is not recommended by the authors or such organisations as USBR (1989a). However, both systems are used successfully by experienced contractors.

Care must also be taken to avoid contamination where haul roads cross the filter zones. This can be facilitated by covering the filter with a geotextile or geomembrane

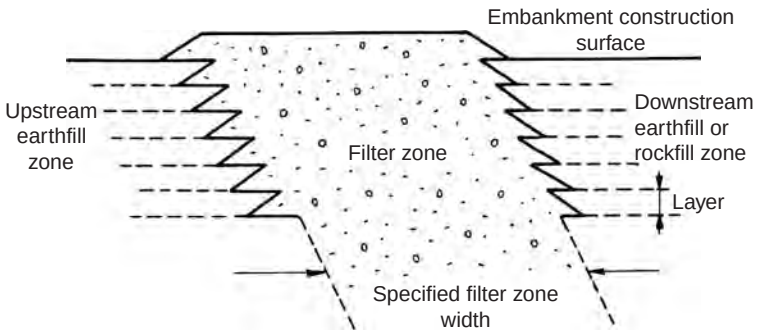


Figure 9.30 Filter zone placement ahead of other zones which is generally desirable.

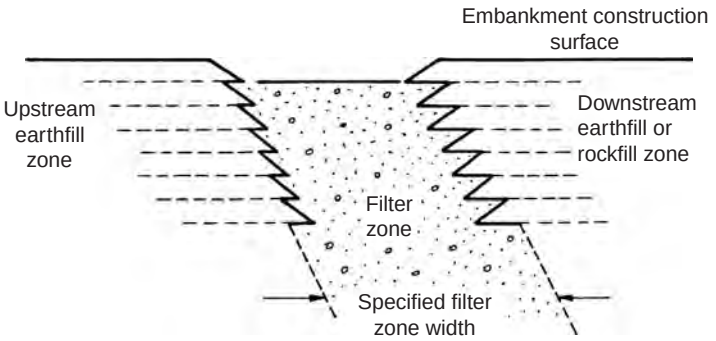


Figure 9.31 Filter zone following construction of other zones which is generally undesirable.

before building the haul road. If practicable, it is wise to shape the surface of the embankment fill, so that surface water flows away from the filter.

9.5.3 Compaction of filters

The general objectives for the compaction of filter zones are:

(a) Filters upstream and downstream of the dam core:

- Sufficient compaction to ensure the filter shear strength does not influence the stability of the embankment.
- For saturated filters, compaction should be to a relative density (density index) such that flow liquefaction under earthquake is not possible, i.e. a relative density ≈70% is required.
- The deformation modulus of the filter should ideally be compatible with the adjacent core material and rockfill. However in practice dense filters are likely to have higher moduli than rockfill or earthfill.
- Avoid excessive breakdown of the filter materials by the compaction equipment.

Table 9.14 Typical filter compaction specifications.

Zone and location in embankment	*Standards Specification*	*Method Specification*
Zone 2A horizontal filter drain	AS 1289 5.6.1, Density index > 70% ASTM D 1556, Relative density > 70%	3 passes of a vibratory roller with static weight not less than 8 tonnes static weight and a centrifugal force not less than 240 kN. Maximum layer thickness 400 mm after compaction
Zone 2A inclined or vertical filter drain	AS 1289 5.6.1, Density index > 70% ASTM D 1556, Relative density > 70% Method specification adequate and preferred	2 passes of a vibratory roller with static weight not less than 8 tonnes static weight and a centrifugal force not less than 240 kN. Maximum layer thickness 400 mm after compaction
Zone 2B horizontal drain	Not applicable. Zone 2B too coarse to do density index/ relative density tests	3 passes of a vibratory roller with static weight not less than 8 tonnes static weight and a centrifugal force not less than 240 kN. Maximum layer thickness 500 mm after compaction
Zone 2B inclined or vertical drain	Not applicable. Zone 2B too coarse to do density index/ relative density tests	2 passes of a vibratory roller with static weight not less than 8 tonnes static weight and a centrifugal force not less than 240 kN. Maximum layer thickness 500 mm after compaction

(b) Horizontal or near horizontal filters:

- The compaction should be to a relative density (density index) such that flow liquefaction under earthquake is not possible, i.e. a relative density of $\approx 70\%$ is required.
- Avoid excessive breakdown of the filter materials by the compaction equipment.

Filters should be compacted in layers using a vibratory smooth steel drum roller. Filters are usually well-graded granular materials and are readily compacted to a dense condition. Hence for the majority of dams, a "methods" type specification for compaction is the most practicable, i.e. a maximum layer thickness, coupled with a number of passes of a vibratory roller of a specified static weight and centrifugal force.

Table 9.14 gives some examples of "standards" type specifications, where a density index (relative density) is specified. This is not the preferred basis for the routine (daily) control of placement on the dam because of the cost involved in carrying out the tests, the delay in the results and the difficulty of carrying out the testing required, particularly in Zone 2B filters. Zone 2B filters will commonly have gravel up to 50 mm or 75 mm size, requiring a density in place test hole at least 200 mm or 300 mm diameter and laboratory compactions in non standard 300 mm or 500 mm diameter cylinders.

If a minimum density index is specified, there should be trials early in the construction to define a method which will give the required compaction and with daily control by the method specification. In this case a requirement to compact to a density index

of say 70% or 80% would be appropriate. It should be noted that requiring a density index greater than say 80% is likely to result in excessive breakdown of filters under the compactive effort and will achieve little or no benefit. There are some arguments, e.g. Sherard (1985a), that over-compaction can lead to excessively high moduli for the filters, resulting in their settling less than the dam core during construction and first filling and thereby reducing vertical stresses and horizontal stresses in the core. This can foster the development of hydraulic fracture. Some therefore favour minimal compaction of these filters – e.g. 1 or 2 passes of a 10 tonnes steel drum roller, using layer thickness of 500 mm. It is arguable whether it is practicable to set an upper limit on density index in dam construction, such as for Peter Faust dam in Table 9.14, other than as a guide to contractors. It would seem difficult to insist on removal of over-compacted material without raising the issue of contractual claims.

9.6 USE OF GEOTEXTILES AS FILTERS IN DAMS

9.6.1 Types and properties of geotextiles

A wide range of different geotextiles is available. These differ in the type of polymer used for manufacture, the type of fibre construction and how this fibre is manufactured into a fabric.

Giroud and Bonaparte (1993) describe the different types of geotextiles as follows:

Woven geotextiles are composed of two sets of parallel yarns systematically interlaced to form a planar structure. Generally, the two sets of yarns are perpendicular. The manner in which the two sets of yarns are interlaced determines the weave pattern. By using various combinations of three basic weave patterns, i.e. plain, twill, and satin, it is possible to produce an almost unlimited variety of fabric constructions. Woven geotextiles can be classified according to the type of yarn, as discussed below:

- Multifilament woven geotextiles are made with yarns that comprise many filaments (typically 100 or more).
- Monofilament woven geotextiles are made with yarns that are single filament.
- Slit film woven geotextiles are obtained by weaving small tapes produced by slitting a plastic film with blades.
- Fibrillated woven geotextiles are obtained by weaving yarns that are bundles of tapelike fibres still partially attached to each other. Such bundles are film strips that have been nicked and broken up into fibrous strands.

Knitted geotextiles include two types: the classical knits and the insertion knits.

- Classical knits are formed by interlocking a series of loops of one or more yarns to form a planar structure. The way the loops are interlocked identifies the type of knit, such as jersey. The knitted geotextiles of the classical knit type are highly deformable materials;
- Insertion knits are produced by inserting yarns into a knit that is being made. Using this manufacturing process, it is possible to make geotextiles that combine properties of woven fabrics, and classical knits.

Nonwoven geotextiles are formed from continuous filaments or from short fibres (also called staple fibres), arranged in all directions (but not necessarily at random) and bonded together into a planar structure. Synthetic filaments are produced by extrusion and are, therefore, continuous, whereas synthetic fibres are obtained by cutting filaments into lengths of 100–150 mm (4–6 inches). Natural fibres are generally short. For nonwoven geotextile, the continuous filaments or short fibres are first arranged into a loose web then bonded together using one, or a combination, of the following processes: chemical bonding, thermal bonding and mechanical bonding by needle-punching.

Chemically bonded nonwoven geotextiles are obtained by bonding the filaments or fibres together using a cementing medium such as glue, rubber, latex, cellulose derivative or, more frequently, synthetic resin.

Heat bonded nonwoven geotextiles are obtained by bonding the filaments or fibres together using heat. When heat is provided by heated rollers that press the fabric, the resulting heat bonded nonwoven geotextile is relatively stiff and thin, typically 0.5–1 mm.

Needle-punched non-woven geotextiles are obtained by entangling the filaments or fibres together using thousands of small barbed needles, which are set into a board and punched through the fabric and withdrawn many times as the fabric passes under the board. Needle-punched non-woven geotextiles are relatively thick, typically 1–5 mm.

The three bonding processes described above can be combined. For example, needle-punched non-woven geotextiles are typically compliant, but, if they are exposed to controlled heat on one side, they become stiffer on that side while remaining compliant on the other side.

Multilayer geotextiles are produced by bonding together several layers of fabrics. Two techniques for manufacturing multiplayer geotextiles are stitch bonding and needle-punching. Stitch bonding consists of stitching together several layers of fabric. This is often used to produce high strength geotextile from slit film woven fabrics. The needle-punching process (described above for non-woven geotextile manufacturing) is used, for example, to associate woven and needle-punched non-woven fabrics.

Yarns, filaments, fibres, and tapes are usually made from polypropylene or polyester. Other polymers used include polyamide (commercially known as Nylon), polyethylene and polyamide (commercially known as Kevlar). Some natural fibres such as jute and coconut fibres are also used.

There are also other geotextile related products such as geomats, geonets, geogrids and geocells. These are described in Giroud and Bonaparte (1993), FEMA (2008).

When considering the use of geotextiles as filters in dam construction, the important properties are the "particle opening size" that affects its ability to act as a filter and its "permeability", or ability to allow water to pass through. The "particle opening size" of the geotextile is usually measured by sieving a standard soil or single sized glass spheres on the geotextile for a fixed period and observing the quantity and particle size of the soil which passes through. There is no one accepted standard and different methods use dry or wet sieving and consider the geotextile size as that allowing zero, 2% or 5% or 50% of the soil passing.

Koerner (2005) and most authors suggest the use of the apparent opening size AOS (also known as equivalent opening size EOS). The test uses known size glass beads and determines by sieving using successively finer beads, that size of bead for which 5% or

Table 9.15 Some examples of opening sizes and flow rates for geotextiles.

Product name	*Structure woven (W)*	*Opening Size (mm)*	*Permittivity (sec^{-1})*
Terram T4500	NW	0.04–0.08 (O_{50})	n/a
Polyfelt TS50	NW Needle P	0.10 (O_{90})	n/a
Polyfelt TS80	NW Needle P	0.09 (O_{90})	n/a
Bidim A44	NW Needle P	0.08 (O_{95})	1.1
Bidim A64	NW Needle P	0.08 (O_{95})	0.8
Terrafix 400R	NW	0.15 (O_{95})	1.0

less pass the fabric. The AOS or EOS is the US standard sieve number of this size bead. It is also, more commonly, quoted as the equivalent sieve opening in millimetres or 95% opening size O_{95}. This is the form used for most filter design criteria as discussed below. Many manufacturers provide data on the 95% opening size in their product literature and it appears to have become the industry standard, at least for the time being. However for important designs, given the dependence of the D_{95} (or O_{90}, O_{50}) on the test method, caution should be exercised in using the manufacturers' values, unless the test method is compatible with the design method.

Some examples of values are given in Table 9.15.

In most cases, the ability of the geotextile to transmit water across the fabric is also important. This is assessed as the permittivity

$$\psi = \frac{\mathrm{kn}}{\mathrm{t}} \tag{9.2}$$

where
ψ = permittivity
kn = cross plane permeability coefficient
t = geotextile thickness at the normal pressure on the geotextile.

In Darcy's Law:

$$\mathrm{q} = \mathrm{ki\,A} \tag{9.3}$$

$$= \mathrm{kn}\frac{\Delta \mathrm{h}}{\mathrm{t}} \cdot \mathrm{A} \tag{9.4}$$

So

$$\frac{\mathrm{kn}}{\mathrm{t}} = \psi = \frac{\mathrm{q}}{(\Delta \mathrm{h})\mathrm{A}} \tag{9.5}$$

where
Q = flow rate
Δh = head loss across the geotextile
A = area of flow (i.e. of geotextile).

As pointed out by Bertacchi and Cazzuffi (1985) and others, the permittivity is dependent on the normal stress applied to the geotextile, i.e. the ability to transmit

water is reduced as the geotextile is placed under stress which compresses the geotextile and the values adopted for design should account for this. The permittivity is also affected by clogging of the geotextile by fine soil. This is discussed further below.

In some applications the geotextile may be used to transmit water along its plane, i.e. it is in itself performing a drainage function. This is measured by transmissivity θ and is given by:

$$\theta = k_p t = \frac{qL}{\Delta h W} \tag{9.6}$$

where
θ = transmissivity
k_p = permeability in the plane of the fabric
t = thickness of the fabric
q = low rate in the plane of the fabric
L = length of the fabric
W = width of fabric
Δh = head lost.

As shown in Figure 9.32, transmissivity is also affected by the applied normal stress.

9.6.2 Geotextile filter design criteria

9.6.2.1 *General requirements*

The requirements for the design of geotextile filters are:

- Prevent erosion of soil particles from the base soil they are protecting; i.e. act as a filter.
- Allow drainage of seepage water without significant flow impedance.
- Not clog excessively with particles from the soil they are protecting.
- Be sufficiently strong to resist damage during construction.
- Be sufficiently durable to resist adverse ultraviolet light and chemical exposure during construction and in service.

9.6.2.2 *Filtering requirement*

There are many different filter design criteria for geotextiles. Summaries are given in Bertacchi and Cazzuffi (1985), ICOLD (1986) and Christopher *et al.* (1993). Critical reviews are given in Giroud (1996) and Heerten (1993). Palmeira and Fannin (2002) give a more recent review but they do not recommend a particular method.

There appear to be two methods which are widely used in USA:

(a) The method of Dr. Giroud and co-workers, as described in Luettich *et al.* (1992). This is recommended by FEMA (2008) and is the design method used by manufacturers such as Mirafi. It was originally described in Giroud (1982). The dynamic loading criteria for this method are from Heerten (1984).

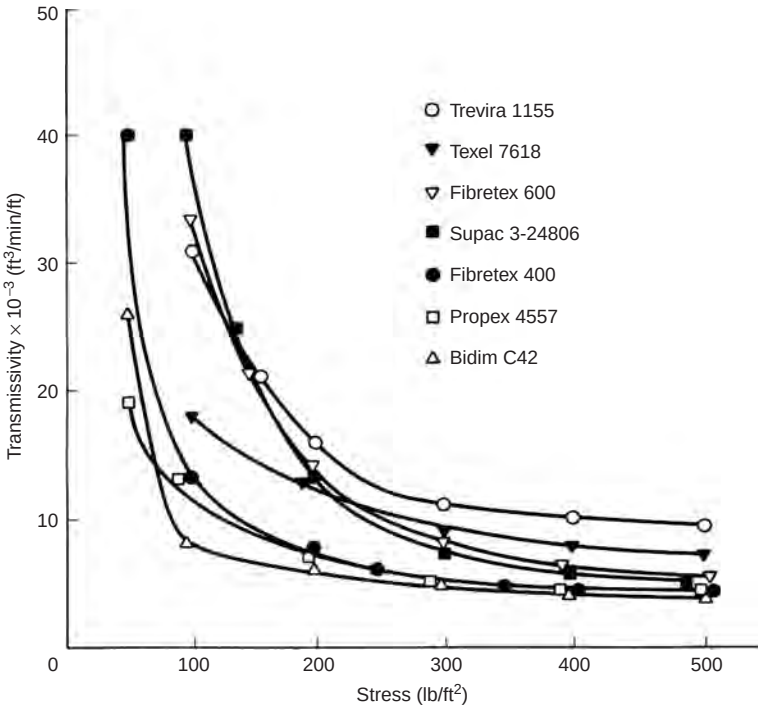


Figure 9.32 Transmissivity versus applied normal stress for various needle punched non-woven geotextiles (Koerner, 1986, reproduced with permission of Pearson Education).

(b) The method of Federal Highways Administration, which was developed by R.D. Holtz, B.R. Christopher and R.R. Berg as described in FWHA (2008). This is recommended by FWHA.

Figures 9.33(a) and 9.33(b) summarize the Luettich *et al.* (1992) method for steady state and dynamic flow conditions. They indicate that standard dewatering drains, wall drains, and leachate collection drains are examples of steady state flow conditions. Shoreline and coastal embankment protection layers are typical examples where waves and water currents may cause dynamic conditions.

The FWHA (2008) recommended method is:

Steady State Flow Conditions

$$O_{95} < BD_{85B} \tag{9.7}$$

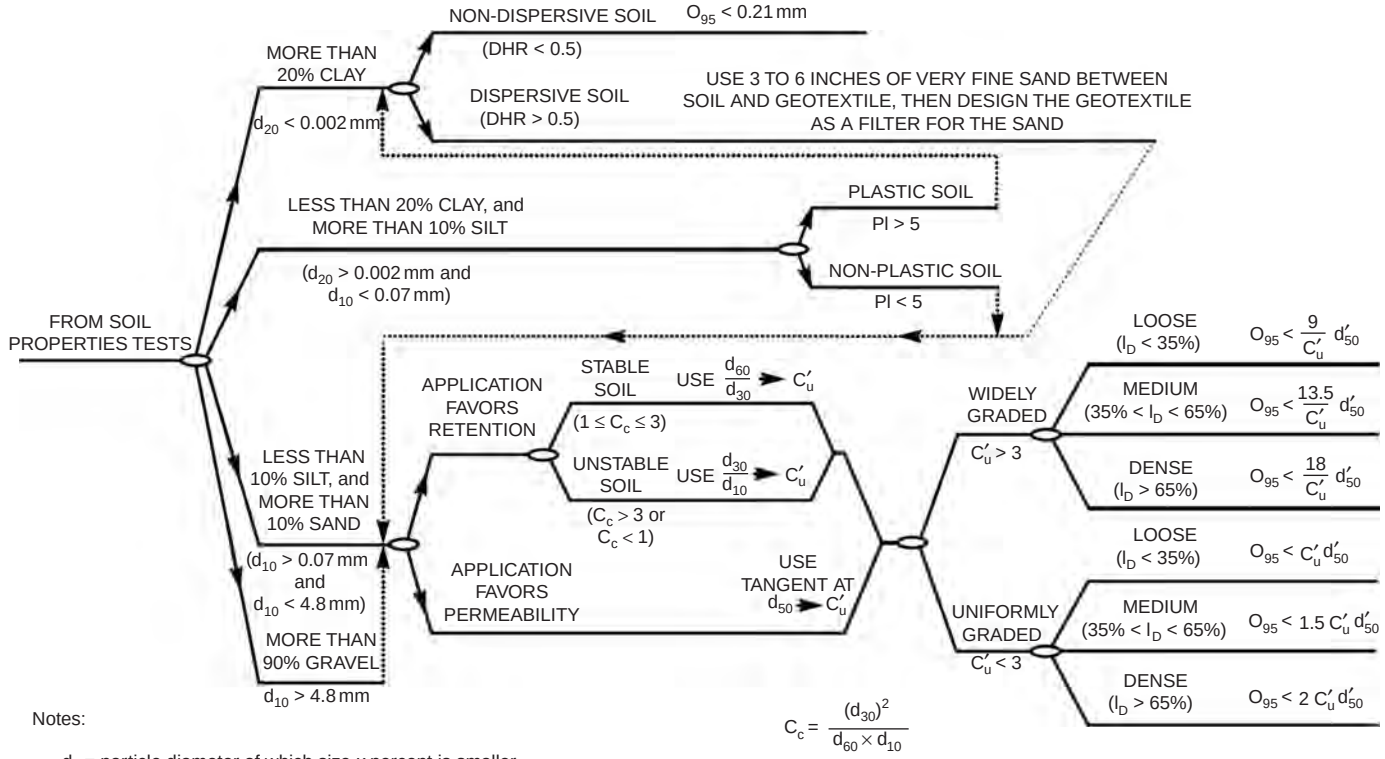
where:

O_{95} = opening size in the geotextile for which 95% are smaller (mm);

B = a coefficient (dimensionless); and

D_{85B} = soil particle size for which 85% are smaller (mm).

Chart 1. Soil retention criteria of steady-state flow conditions



Notes:

d_x = particle diameter of which size *x* percent is smaller

$C_u' = \sqrt{\frac{d_{100}'}{d_0'}}$ where: d_{100}' and d_0' are the extremeties of a straight line drawn through the particle-size distribution, as directed above and d_{50}' is the midpoint of this line

$C_c = \frac{(d_{30})^2}{d_{60} \times d_{10}}$

I_D = relative density of the soil

PI = plasticity index of the soil

DHR = double-hydrometer ratio of the soil

O_{95} = geotextile opening size

Figure 9.33(a) Soil retention criteria for steady-state flow conditions (Luettich *et al.*, 1992).

Chart 2. Soil retention criteria of dynamic flow conditions

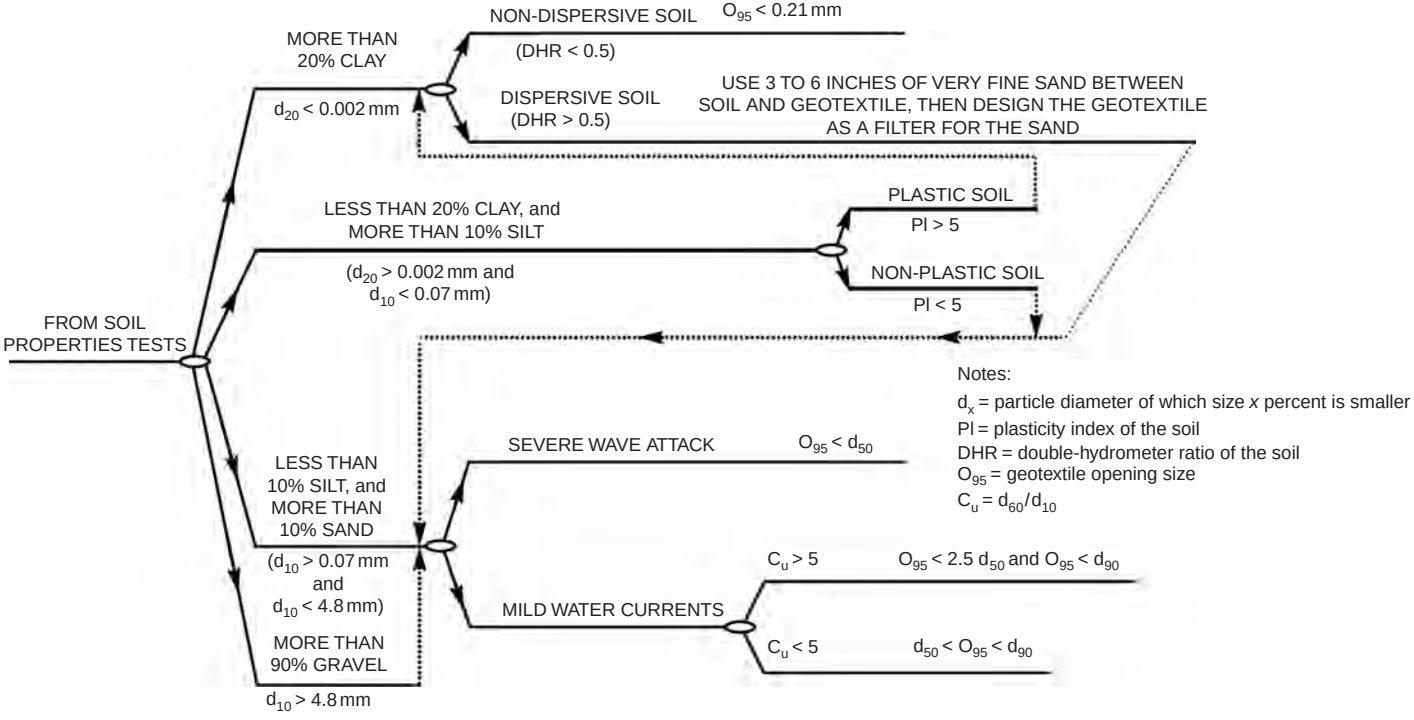


Figure 9.33(b) Soil retention criteria for dynamic flow conditions (Luettich *et al*., 1992).

The coefficient B ranges from 0.5 to 2 and is a function of the type of soil to be filtered, its density, the uniformity coefficient Cu if the soil is granular, the type of geotextile (woven or nonwoven) and the flow conditions.

For *sands, gravelly sands, silty sands,* and *clayey sands* (soils with less than 50% passing the 0.075 mm sieve B is a function of the uniformity coefficient, Cu:

$$\mathrm{Cu} < 2 \text{ or } > 8 : \mathrm{B} = 1 \tag{9.8a}$$

$$2 < \mathrm{Cu} < 4 : \mathrm{B} = 0.5\mathrm{Cu} \tag{9.8b}$$

$$4 < \mathrm{Cu} < 8 : \mathrm{B} = 8/\mathrm{Cu} \tag{9.8c}$$

where:

$$\mathrm{Cu} = \mathrm{D}_{60}/\mathrm{D}_{10}.$$

If the protected granular soils contain appreciable fines, use only the portion passing the No. 4 (4.75 mm) sieve for selecting the geotextile.

For *silts and clays* (soils with more than 50% passing the No. 200 (0.075 mm) sieve), B is a function of the type of geotextile:

$$\text{For woven geotextiles, } \mathrm{B} = 1;\ \mathrm{O}_{95} < \mathrm{D}_{85\mathrm{B}} \tag{9.9a}$$

$$\text{For nonwoven geotextiles, } \mathrm{B} = 1.8;\ \mathrm{O}_{95} < 1.8\mathrm{D}_{85\mathrm{B}} \tag{9.9b}$$

$$\text{and } \textit{for both}, \ \mathrm{O}_{95} < 0.3 \text{ mm} \tag{9.9c}$$

Dynamic Flow Conditions

$$\mathrm{O}_{95} < 0.5\mathrm{D}_{85\mathrm{B}} \tag{9.10}$$

Degoutte and Fry (2002) indicate that the French Committee on Large Dams recommends the French Standard method as described in AFNOR (1993). This recommends:

$$\mathrm{O}_{95} < \mathrm{CD}_{85\mathrm{B}} \tag{9.11}$$

where

$$\mathrm{C} = \mathrm{C}_1 \times \mathrm{C}_2 \times \mathrm{C}_3 \times \mathrm{C}_4 \tag{9.12}$$

$C_1 = 1$ if $C_u \geq 5$
$= 0.8$ if $C_u < 5$
$C_2 = 1.25$ for plastic soils compacted to $\geq 95\%$ density ratio
$= 0.8$ for plastic soils compacted to $<95\%$ density ratio
$= 1.25$ for non-plastic soils compacted to $\geq 50\%$ density index
$= 0.8$ for non-plastic soils compacted to $<50\%$ density index
$C3 = 1$ if the gradient in the soil adjacent the filter is ≤ 5
$= 0.8$ if the gradient in the soil adjacent the filter is > 5
$= 0.6$ for alternating gradients
$C4 = 1$ if the geotextile is acting only as a filter
$= 0.3$ if it acts as a filter and drain.

In addition for plastic soils, $O_{95} > 80$ micron (0.08 mm).

Table 9.16 compares the results of use of these filter design methods and the methods of Heerten (1984) and Loudiere *et al.* (1982) with filter designs using the Sherard and Dunnigan (1989) and Foster and Fell (2001) methods. To allow this comparison it has been assumed that O_{95} = D15F/9 which is supported by the following information:

(a) Sherard *et al.* (1984a) who found this based on the results of filter tests on uniform base soils. In a later series of tests (Sherard 1984), materials passing through the filters were caught and gradings of the material showed that approximately 97 to 99% of the particles were finer than $D_{15F}/9$ ($=0.11D_{15F}$). The maximum sizes of the particles ranged from $0.13–0.20D_{15F}$ with a median of about $0.16D_{15F}$.
(b) Kenney *et al.* (1985) obtained similar results and found the maximum possible size of particle that can pass through a filter equal to $D_{5F}/4$ or $D_{15F}/5$ ($=0.20D_{15F}$).
(c) Foster and Fell (1999a, 2001) who carried out continuing erosion filter tests in which the eroded materials that were collected in the drums were sieved to determine the size of particles passing through the filters. These showed that approximately 97–100% of the particles passing through the filter were finer than a particle size equal to $D_{15F}/9$ which was in good agreement with the results of Sherard *et al.* (1984a).

The particle size distributions of the soils in Table 9.16 are shown in Figure 9.34. For soils 1, 2 and 3, two cases were considered, one for the soils being dispersive (Pinhole test D1 or D2; Emerson Crumb test 1 or 2) and the other non-dispersive.

Soil 1 is a Sherard and Dunnigan (1989) Group 1 soil; soil 2a Group 2 soil on the fine side; soil 3a Group 2 soil on the coarse side; soils 4 and 5 Group 3 soils with different uniformity coefficients.

This is not an exhaustive comparison of the methods but it is reasonable to conclude the following:

Luettich, Giroud and Bachus method:

(a) Gives filter designs for non-dispersive Group 1 soils which are too coarse for no-erosion filters, and may even give filters in the continuing erosion range. This is the case even if the dynamic design criteria are used.
(b) Requires that for dispersive Group 1 soils the filter should be granular, not a geotextile. This is as required by Foster and Fell (2001) method.

Table 9.16 Comparison of filter design methods.

	Percent passing size (mm)							
Parameter	*1 ND*	*1 D*	*2 ND*	*2 D*	*3 ND*	*3 D*	*4*	*5*
D95	0.1	0.1	0.85	0.85	3.6	3.6	0.29	1.1
D90	0.075	0.075	0.6	0.6	2.7	2.7	0.28	1.0
D85	0.05	0.05	0.425	0.425	2.36	2.36	0.26	0.9
D60	0.002	0.002	0.06	0.06	0.6	0.6	0.21	0.6
D50	<0.002	<0.002	0.02	0.02	0.2	0.2	0.17	0.425
D30	<0.002	<0.002	<0.002	<0.002	0.006	0.006	0.13	0.2
D10	<0.002	<0.002	<0.002	<0.002	<0.002	<0.002	0.1	0.1
Cu	NA	NA	>30	>30	>300	>300	2.1	6
Cc	NA	NA	NA	NA	NA	NA	0.8	0.66
	Maximum O_{95} opening size for the design method							
Design Method	*1 ND*	*1 D*	*2 ND*	*2 D*	*3 ND*	*3 D*	*4*	*5*
Sherard and Dunnigan (1989)[4]	0.05 (0.45)	0.05	0.077 (0.7)	0.077 (0.7)	0.077 (0.7)	0.077 (0.7)	0.11 (1.0)	0.4 (3.6)
Foster and Fell (2001)[4]								
No erosion	0.05 (0.45)	0.033 (0.3)	0.077 (0.7)	0.055 (0.5)	0.077 (0.7)	0.055 (0.5)	0.11 (1.0)	0.4 (3.6)
Excessive erosion	0.075 (0.7)	0.075 (0.7)	0.6 (5.4)	0.6 (5.4)	0.24 (2.2)	0.24 (2.2)	0.29 (2.6)	1.0 (9.0)
Continuing erosion	0.075 (0.7)	0.075 (0.7)	0.85 (7.6)	0.85 (7.6)	3.6 (32)	3.6 (32)	0.29 (2.6)	1.1 (9.9)
Luettich, Giroud and Bachus (1992)								
Steady state	0.21	GF[5]	0.21	GF	0.21	GF	0.17–0.34	0.63–1.26
Dynamic[6]	0.1	GF	0.1	GF	0.1	GF	0.17	0.425
FHWA (2008)								
Steady state	0.09	0.09	0.3	0.3	2.36	2.36	0.26	1.2
Dynamic	0.025	0.025	0.21	0.21	1.18	1.18	0.13	0.45
Heerten (1984)	0.075	0.075	0.1	0.1	0.1	0.1	0.28	1.0
Loudiere (1982)	0.05	0.05	0.425	0.425	2.36	2.36	0.14	1.1
AFNOR (1993)	0.08	0.08	0.08	0.08	0.08	0.08	0.21	0.9

Notes:
(1) See Figure 9.34 for plots of particle size distribution.
(2) ND = non-dispersive; D = dispersive soil.
(3) For these methods opening size calculated assuming $O_{95} = D15/9$.
(4) Figures in brackets are D15 size.
(5) GF = granular filter required according to design method.
(6) "Severe wave attack" condition assumed which gives the finest EOS.

(c) For Group 2 soils the filter designs are generally in the "some erosion" range; that is between no-erosion and excessive erosion criteria.
(d) For Group 3 soils the filter designs are also generally in the "some erosion" range but if the "dense" soil criteria are used they get into the excessive erosion area.
(e) If the "dynamic" criteria are adopted the filters are still in the "some erosion" range but quite close to no-erosion criteria for Group 2 and 3 soils.

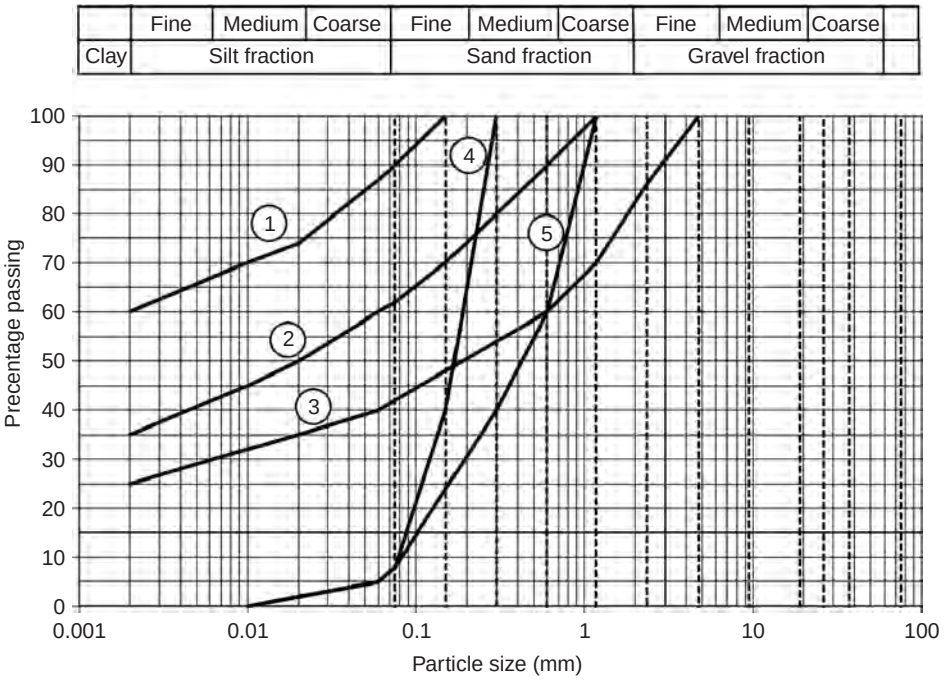


Figure 9.34 Particle size distributions for soils assessed in Table 9.16.

FWHA (2008) method

(a) The "steady state" criteria give filters always coarser than the Luettich *et al.* (1992) method and are therefore too coarse to satisfy Sherard and Dunnigan (1989) no-erosion criteria. For the Group 1 and Group 3 soils modelled it resulted in filters in or close to the continuing erosion range and for the coarser Group 2 soil it gave a filter in the excessive erosion range.
(b) For "dynamic" criteria give filters which are sufficiently fine for Group 1 soils, but as can be seen from Tables 9.15 and 9.16 the required O_{95} is finer than typical generally available geotextile filters.
(c) For Group 2 soils the dynamic criteria give a filter in the "some erosion" range for the finer soil, and in the "excessive erosion" range for the coarser soil.
(d) For Group 3 soils the dynamic criteria give filters in the "some erosion" range but close to no-erosion requirements.

Heerten (1984) method

(a) Gives filters for Group 1 soils potentially in the continuing erosion range.
(b) For Group 2 soils gives a filter in the "some erosion" range but close to no-erosion filters.
(c) For Group 3 soils gave filters at the excessive erosion boundary.

Loudiere et al. (1982) method

(a) Gave a no-erosion filter for the Group 1 gradations modelled, but would be too coarse for Group 1 soils with little or no sand size fraction.
(b) For Group 2 soils gives a filter in the "some erosion" range for the finer soil, and in the "excessive erosion" range for the coarser soil.
(c) For Group 3 soils gives a filter in the "some erosion" range for the finer more uniform soil, and in the "excessive erosion" range for the coarser soil.

French Standard, recommended by French National Committee on Large Dams (AFNOR, 1993)

(a) Gives filter designs for Group 1 soils which are too coarse for no-erosion filters, and may even give filters in the continuing erosion range.
(b) Gives no-erosion filters for Group 1 and 2 soils, except for the finer requirement for dispersive soils recommended by Foster and Fell (1999a, 2001).
(c) Gives some erosion for Group 3 soils.

Overall it is concluded that:

(a) The geotextile filter design methods considered all give some filters coarser than required by no-erosion criteria for granular filters. In particular the "steady state" methods of Luettich *et al.* (1992) and FWHA (2008) give filters which are far too coarse for plastic soils (Group 1 and 2 soils).
(b) It is not clear why this is so but it seems geotextile filter design methods are not seeking no-erosion conditions and may not have envisaged the erosion conditions which occur in a crack in a dam core which has been modelled in the Sherard and Dunnigan (1989) and Foster and Fell (2001) methods.
(c) The French standard (AFNOR, 1993) goes closest to satisfying no erosion criteria, and would be the best of the methods to use.
(d) However if no-erosion filters are required it would be wise to design the geotextile filter by estimating the D_{15F} required using Foster and Fell (2001), and converting that to the required O_{95} for the geotextile filter using $O_{95} = D_{15F}/9$.
(e) The finest geotextile filter available are not sufficiently fine for to give no-erosion filters for Group 1 soils (soils with greater than 85% fines passing 0.075 mm) and a granular filter should be provided regardless of whether the soil is dispersive or not. Alternatively a transition zone should be constructed between the Group 1 soil and the geotextile filter.
(f) For Group 2 non-dispersive soils the finest geotextile generally available ($O_{95} = 0.080$ mm or 80 micron) just satisfies no-erosion criteria. However for dispersive soils a granular filter should be provided.

9.6.2.3 *Clogging and blinding resistance*

Geotextile filters are prone to clogging and blinding. These are shown in Figure 9.35.

If this occurs the ability of the geotextile to act as a drain may be impaired, and pore pressures upstream of the geotextile filter may be increased.

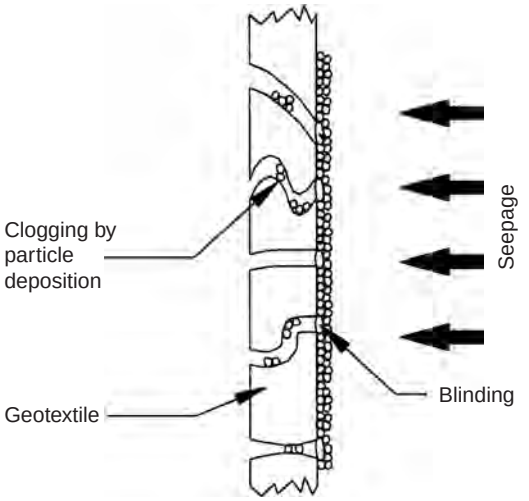


Figure 9.35 Definitions of clogging and blinding (FHWA, 2008).

FWHA (2008) discuss these and suggest that for less critical/less severe applications the geotextile may be designed to allow erosion of finer particles through the filter with a "bridge" of coarser particles forming to provide overall filter performance. This is contrary to good dam design practice for no-erosion filters and is not recommended.

Luettich *et al.* (1992) suggest that to minimize the risk of clogging the following criteria should be met:

- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For non-woven geotextiles, use the largest porosity (n) available, but not less than 30%.
- For woven geotextiles, use the largest open percent area (POA) available, but not less than 4%.

FWHA (2008) suggest a porosity for non-woven geotextiles not less than 50% and point out most have porosities greater than 70%.

FWHA (2008) indicate that for critical/severe conditions performance laboratory tests should be carried out. They and Luettich *et al.* (1992) recommend the gradient ratio test (ASTM D 5105) for soils with a permeability $>10^{-6}$ m/sec, and the hydraulic conductivity ratio test (Williams and Abouzakhm, 1989; ASTM D 5567) for lower permeability soils.

Figure 9.36 shows the test set up for the gradient ratio test. The gradient ratio is the ratio of the head loss across the interface (usually 25 mm of the soil plus geotextile) to the head loss of the upper 50 mm of the soil. They indicate that a Gradient Ratio < 3 is recommended by USACE.

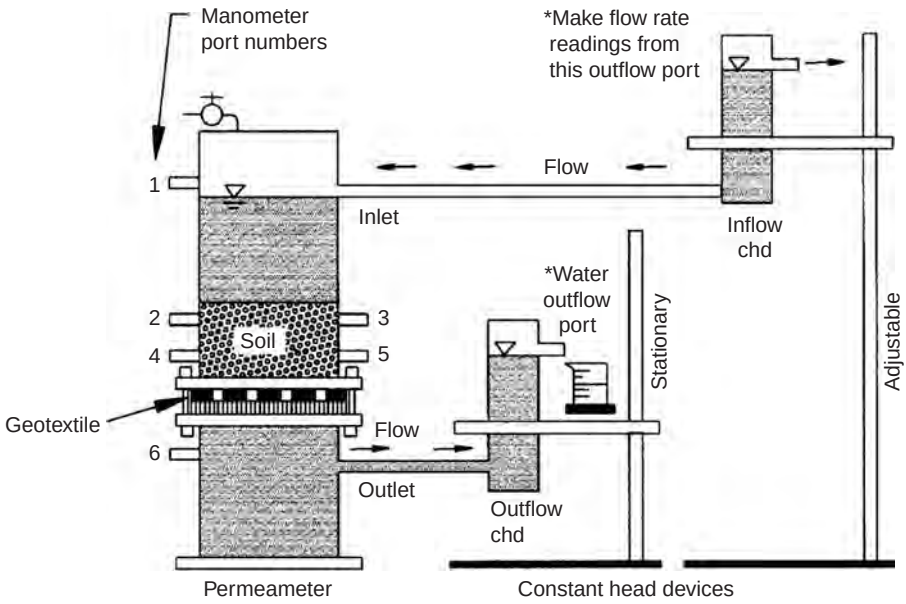


Figure 9.36 Diagram of gradient ratio test device (FWHA, 2008).

The authors have experience of the gradient ratio test for mine tailings and it is a relatively straight forward test to carry out. The geotextile filter manufacturers may also be equipped to do these tests for specific projects.

It should be noted that contrary to the advice in FEMA (2008) the gradient ratio tests does not confirm filter retention for plastic soils because it does not model the concentrated leak condition which is the most common potential internal erosion mode for those soils.

The authors are aware of cases where limonite depositing from seepage water has clogged geotextile filters. Palmeira and Fannin (2002) also warn of chemical and biological clogging.

Luettich *et al.* (1992) and FWHA (2008) suggest biological clogging can be assessed by ASTM D 1987.

FEMA (2008) indicate that the problem of blinding can also occur if there are open voids in the base soil, or if the base soil surface is irregular and thus prevents good contact with the geotextile from being established and maintained. This problem has been observed for geotextile placements against vertical or steeply inclined slopes. Figure 9.37 shows development of the blinding layer.

FEMA (2008) indicate that two precautions are needed to eliminate the tendency towards blinding:

- Use fine gravel (around 25 mm maximum size) rather than a coarse rock for the drainage layer. By limiting the size of the gravel placed in contact with the geotextile, the geotextile will be held tightly against the base soil Giroud (1997).

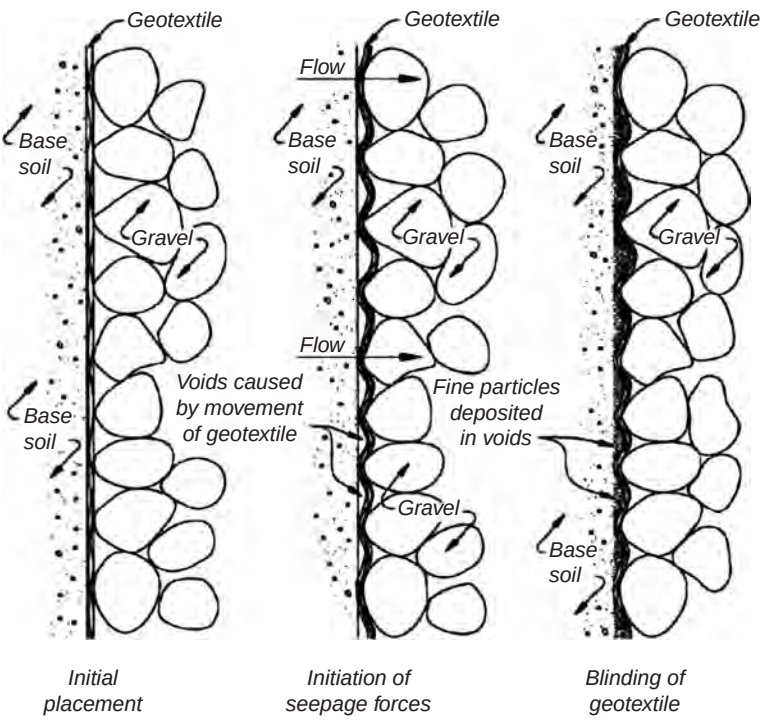


Figure 9.37 Illustration showing the progressive steps leading to blinding of a geotextile caused by using drainage aggregate which is too large in size (FEMA, 2008).

- Ensure the base soil surface is smooth and regular and place a flexible geotextile in close contact with the base soil with a minimum of wrinkles.

Regarding the maximum gravel size to use against the geotextile, published recommendations vary from 19 mm (Giroud, 1997) to 37 mm (Van Zyl and Robertson, 1980).

9.6.2.4 *Permeability requirement*

There are several views on the selection of geotextile permeability including:
Luettich et al. (1992), Giroud (1996),

$$k_G > i_s k_s \tag{9.13}$$

where
k_G = permeability of the geotextile
i_s = hydraulic gradient in the soil adjacent the geotextile (which may be greater than 1)
k_s = permeability of the soil.

Giroud (1996) recommends a large safety factor (e.g. 10) should be used to allow for uncertainties in the permeabilities.

Christopher et al. (1993); FWHA (2008)

$$k_G > 10k_s$$

Loudiere et al. (1982), Degoutte and Fry (2002)

$k_G > 100k_s$, where k_G is the uncompressed geotextile permeability.

Heerten (1984)

$$k_{Gclogged} > k_s \tag{9.14}$$

From these data, it would seem wise to require an uncompressed geotextile permeability of at least $10k_s$ and preferably $100k_s$. A check against the results of gradient ratio tests would also be appropriate.

9.6.2.5 *Durability or "survivability" requirement*

There are two aspects to consider:

(a) Durability during installation – geotextiles are prone to tearing and puncturing as they are being placed, or by materials being compacted on top of or adjacent to them; They may also be subject to degradation by sunlight.
(b) Durability during operation, both from a viewpoint of mechanical issues, e.g. ability to cope with differential deformations, and chemical durability.

Table 9.17 lists the AASHTO (1996) minimum requirements which are referred to by Luettich *et al.* (1992), FWHA (2008) and FEMA (2008).

Luettich *et al.* (1992) indicate that durability to ultraviolet light can be assessed by ASTM D 4355, and chemical compatibility using USEPA (1987).

The author's preference and experience is that for dam and levee applications needle punched non woven geotextile should be used. These have a greater thickness, are reasonably robust, and the available non-woven geotextiles have some of the finer O_{95}. FEMA (2008) indicate these are most likely the best for most filtration applications.

9.6.2.6 *Use of geotextile filters in dams*

Use of geotextiles to perform a permanent function, probably in lieu of a sand/gravel filter, does require consideration of the effectiveness of the geotextile as a "filter" and "drain" as has been discussed in preceding Sections. It also requires consideration of long-term function without damage due to deformation or clogging and durability, particularly if the geotextile is performing a critical function and is not accessible for checking its condition or for repair. The following summarizes some views on the use of geotextiles in dams.

Table 9.17 Survivability strength requirements for geotextiles (AASHTO, 1996).

Application	*Contact Stress*	*Grab Strength N ASTM D 4632*	*Elongation % ASTM D 4632*	*Sewn Seam Strength N ASTM D 4632*	*Puncture Strength N ASTM D 6241*	*Burst Strength N ASTM D 3786*	*Trapezoid Tear N ASTM D 4533*
Subsurface Drainage	High Stress – angular drainage media, heavy compaction, high confining stress	800	–	710	360	2000	220
Subsurface Drainage	Low Stress – rounded drainage media, light compaction, low confining stress	360	–	310	110	900	110
Armoured Erosion Control	Direct stone placement	890	15	800	360	2200	220
Armoured Erosion Control	Sand cushion, low drop height	400	15	360	180	960	130

ICOLD (1986) outlines possible uses of geotextiles in dam construction. They conclude that:

- Considerable caution is required in using geotextiles to prevent erosion at interfaces which are subject to seepage from the reservoir (i.e. location "g" in Figure 9.2).
- Successful use in non critical locations, e.g. under slope protection (as in "c" in Figure 9.2) should not be used as justification for use in critical areas.
- Geotextiles are subject to damage during construction, particularly from angular coarse-grained materials. This may have little effect on their use as a separator of different soils during construction but continuity is necessary for effectiveness as a filter.
- Geotextiles, once buried, have similar durability to other man-made materials. However they are subject to rapid deterioration if left exposed to sunlight before, during or after construction. (The authors add that if polyester geotextiles are placed next to concrete, particularly where the concrete may be heated by sunlight, the polyester can degrade under the high pH conditions. These geotextiles can be used as separators under concrete, as during the placement they will stop the paste in the mix from being lost, but they would be assumed as playing no further role as a filter thereafter.)

ICOLD (1994) quote Roth and Schneider (1991) as concluding: "The availability of geosynthetics presents the dam engineering community with both opportunities and challenges – opportunities to design economical, safe dams and challenges to avoid inadvertently committing an unfortunate error leading to a dam failure. Blanket avoidance of geosynthetics is inappropriate, since they may permit safe, more economical designs in some cases. On the other hand, the potential for catastrophic damage and loss of life in the event of a major dam failure suggests a cautious approach and prudent use of geosynthetics in dams".

Roth and Schneider (1991) offer the following suggested broad guidelines for the use of geosynthetics in dams in the hope of generating open and productive discussion by the dam engineering community on the appropriate uses of geosynthetics in modern dam design:

(1) Geosynthetics should not be used in a configuration where they serve as the sole defence against dam failure. For example, if a geomembrane is used as the sole water barrier within an embankment, the remainder of the embankment design should be checked to assure that the embankment would be completely safe if portions of the geomembrane were suddenly and completely removed.
(2) Geosynthetics should primarily be used where they can be readily exposed, repaired, or replaced. For example, a geomembrane used on the face of a concrete dam can be examined and repaired or replaced if required.
(3) We should expand our use of geosynthetics when they prove to be economically attractive in non-critical, redundant or superficial applications. In this way we can expand our knowledge of the behaviour of geosynthetics in dams and gain additional, valuable experience on the long-term performance of geosynthetics in service conditions.

While not endorsing these views directly, ICOLD (1994) appears to do so indirectly.

FEMA (2008) present a summary of the policy of use of geotextiles in embankment dams in the USA. They indicate that "some geotextile applications such as a protective cushion for a geomembrane, or as a separator/filter beneath riprap are well accepted in the U.S. dam engineering community. Other geotextile applications such as internal filtration, drainage, or as a crack stopper, are controversial and considered to be outside standard engineering practice in the United States for embankment dams. There is a continuing interest in expanding the use of geotextiles in U.S. dam construction and rehabilitation because of the significant cost savings potential; however; economic concerns cannot be allowed to compromise dam safety. The reliability of geotextiles as internal filters or drains in a dam embankment has not been sufficiently established".

FEMA (2008) indicates that: "It is the policy of the National Dam Safety Review Board that geotextiles should not be used in locations that are critical to the safety of the dam". (The National Dam Safety Review Board provides the Director of the US Federal Emergency Management Agency (FEMA) with advice in setting dam safety priorities and considers the effects of national policy issues affecting dam safety. Review Board members include FEMA, the Chair of the Board, and representatives from four Federal Agencies that serve on the interagency Committee on Dam Safety, five State Dam Safety Officials, and one member from the private sector).

FEMA (2008) goes on to summarize the policy of USA Federal Agencies and private organisations. In summary these support the view that geotextiles should not be used as critical filters in embankment dams.

The following objections to the use of geotextiles for filtration and drainage have been put forth by the dam engineering community in the United States (ASDSO, 2003; and from the FEMA, 2008 summary):

- A geotextile fabric may clog when water containing soil in suspension enters the filter face. Sand filters support the soil discharge face and prevent movement of fines that would clog the filter.
- Geotextile fabrics do not support the soil discharge face as a granular filter does. Fabric needs to have intimate contact with the soil discharge face with distance between contact points similar to a granular filter, or soil particle movement will occur clogging the fabric. Coarse granular fill does not provide uniform pressure with close contact points as does a sand filter.
- When used inside the dam, fabrics will have very large soil pressures on both sides of the fabric that will hold it firmly in place with no chance to distribute stresses that are produced by differential movement within the soil mass along the plane of the fabric. When a crack occurs in the dam, it may tear the fabric in the plane of the crack.
- Geotextiles are easily damaged from equipment passing over the material, from protrusions in the underlying material or from moving sheets of the fabric over a rough surface. Damage may not always be detected. Structural integrity of the dam is dependent on complete continuity of the filter drainage zone and when constructed with a fabric, it must be without holes, tears or defects.
- Self healing characteristics of granular filters are not inherent in geotextile materials.
- Failure of rip-rap or armouring armour failure by sliding that has been attributed to clogging (blinding) and pressure build-up beneath the geotextile from wave action. (The authors have been advised of similar problems in river bank erosion protection works).
- Many smaller applications have experienced loss of drainage efficiency in toe drain applications.

The current authors' views mirror those of Roth and Schneider (1991) and ICOLD (1986, 1994). We do not recommend the use of geotextiles in critical locations within dams. We are concerned at the high likelihood of damage during installation or by differential movements of the dam during and after construction, clogging and hence loss of ability to pass the seepage flows through the geotextile and long-term chemical durability. We have been in agreement to the use of geotextiles as:

- As a separator/filter between embankment material and a layer of riprap placed on the upstream slope of low consequence of failure dams, or in a downstream discharge area where failure would not create a hazard to the dam.
- As a filter zone between riprap used to line watercourses and minor spillways in low consequence of failure dams and the underlying foundation soils.

- As a filter in trench drains downstream of the toe of the dam where the coarse drainage layer does not meet filter requirements for the foundations soils.
- As a filter in coffer dams or other temporary structures where the acceptable risks of failure are higher than for dams in general.

However our preference is to rely on granular filters where practicable.

9.6.2.7 *Construction factors*

See FEMA (2008) for information relating to good construction practices for geotextiles.

9.6.2.8 *Sources of detailed information*

There is excellent information in FWHA (2008) and FEMA (2008). Those proposing to use geotextiles should read through these as they contain extensive and practical information.

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Chapter 10

Embankment dams, their zoning and design for control of seepage and internal erosion and piping

10.1 HISTORIC PERFORMANCE OF EMBANKMENT DAMS AND THE LESSONS TO BE LEARNED

Much of the history of development of embankment dams can be related to failures and accidents.

ICOLD (1974) define failures accidents and incidents as follows:

Accident: Three categories of accidents are listed by ICOLD:

Accident Type 1 (A1): An accident to a dam which has been in use for some time but which has been prevented from becoming a failure by immediate remedial measures, including possibly drawing down the water.

Accident Type 2 (A2): An accident to a dam which has been observed during the initial filling of the reservoir and which has been prevented from becoming a failure by immediate remedial measures, including possibly drawing down the water.

Accident Type 3 (A3): An accident to a dam during construction, i.e. settlement of foundations, slumping of side slopes etc., which have been noted before any water was impounded and where the essential remedial measures have been carried out, and the reservoir safety filled thereafter.

Failure: Collapse or movement of part of a dam or its foundation, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people or property downstream (ICOLD, 1995). Incidents to dams during construction are only considered as failures when large amounts of water were involuntarily released downstream.

Two categories of failures are listed by ICOLD (1974):

Failure Type 1 (F1): A major failure involving the complete abandonment of the dam.

Failure Type 2 (F2): A failure which at the time may have been severe, but yet has permitted the extent of damage to be successfully repaired, and the dam brought into use again.

Incident: Either a failure or accident, requiring major repair.

Table 10.1 Statistics of failures and accidents by internal erosion and piping of embankment dams up to 1986 (adapted from Foster *et al.*, 1998).

	Average probability during life of dam ($\times 10^{-3}$)			
	Piping in embankment		*Piping in foundation*	
Zoning category	*Failures*	*Accidents*	*Failures*	*Accidents*
Homogeneous earthfill	16.0	9.2	3.0	11.2
Earthfill with horizontal filter	1.5	0.6	–	3.9
Earthfill with rock toe	8.9	8.0	7.0	3.9
Zoned earthfill	1.2	2.4	0.4	4.6
Zoned earth and rockfill	1.2	7.3	–	7.6
Central core earth and rockfill	<1.1	22.0	–	9.8
Concrete face earthfill	5.3	2.4	10.4	5.8
Concrete face rockfill	<1	3.5	–	–
Puddle core earthfill	9.3	20.7	–	–
Concrete core wall earthfill	<1	8.1	11.8	4.9
Concrete core wall rockfill	<1	21.6	–	–
Hydraulic fill	<1	32.4	15.7	91.8
Zoning unknown	–	–	–	–
All embankment types	3.5	26.8	1.7	6.2

ICOLD (1974, 1995), carried out extensive surveys on dam incidents and from this developed some overall statistics.

Foster *et al.* (1998, 2000a, 2000b), used the ICOLD data and information on the dams which had experienced incidents to develop historic performance data for dams constructed up to 1986.

They showed that about 1 in 25 large embankment dams constructed before 1950 and about 1 in 200 of those constructed after 1950 failed. Table 8.1 in Chapter 8 summarises the statistics of failure of embankment dams during operation (i.e. excluding failures during construction), showing that after overtopping, internal erosion and piping is the most important mode. Slope instability accounts for only 5.5% of failures and only 1.6% for dams built after 1950. Their data shows that for dams in Australia, USA, Canada and New Zealand designed and constructed after 1930, that is, generally after civil engineering design had begun to be put on a more rational process, about 90% of failures were related to internal erosion and piping.

If two failures of hydraulic fill dams and two of unknown zoning are excluded, only 1 in 5000 dams have failed by instability. However 1 in 200 has experienced an instability accident. This reflects the fact that it is possible to reliably assess the factor of safety of a dam, that modern criteria for acceptable factors of safety are reasonably conservative, and that most dams will show excessive deformation if they are marginally stable, allowing remedial works or lowering of the reservoir water level before failure occurs.

Tables 8.2 to 8.6 and Tables 10.1 and 10.2 present additional statistics on internal erosion and piping failures and accidents.

In Table 10.2, the annual probabilities have been calculated allowing for the estimated total years of dam operation up to 1986.

Table 10.2 Historical average probabilities of failure of embankment dams by internal erosion and piping (Foster *et al.*, 1998, 2000a).

	Embankment			*Foundation*			*Embankment into foundation*		
		Average annual P_e ($\times 10^{-6}$)			*Average annual P_f ($\times 10^{-6}$)*			*Average annual P_{ef} ($\times 10^{-6}$)*	
Zoning category	*Average P_{Te} ($\times 10^{-3}$)*	*First 5 years operation*	*After 5 years operation*	*Average P_{Tf} ($\times 10^{-3}$)*	*First 5 years operation*	*After 5 years operation*	*Average P_{Tef} ($\times 10^{-3}$)*	*First 5 years operation*	*After 5 years operation*
Homogeneousearthfill	16	2080	190						
Earthfill with filter	1.5	190	37						
Earthfill with rock toe	8.9	1160	160						
Zoned earthfill	1.2	160	25						
Zoned earth and rockfill	1.2	150	24						
Central core earth and rockfill	(<1.1)	(<140)	(<34)						
Concrete face earthfill	5.3	690	75	1.7	255	19	0.18	19	4
Concrete face rockfill	(<1)	(<130)	(<17)						
Puddle core earthfill	9.3	1200	38						
Earthfill with corewall	(<1)	(<130)	(<8)						
Rockfill with corewall	(<1)	(<130)	(<13)						
Hydraulic fill	(<1)	(<130)	(<5)						
All dams	*3.5*	*450*	*56*	*1.7*	*255*	*19*	*0.18*	*19*	*4*

1. P_{Te}, P_{Tf} and P_{Tef} are the average probabilities of failure over the life of the dam.
2. P_e, P_f and P_{ef} are the average annual probabilities of failure.

There are a number of points which can be seen from this data:

(a) About 1 in 300 embankment dams have failed by piping through the embankment. The most likely to fail are homogeneous earthfill, earthfill with a rock toe and all dams with little or no filters or seepage control zoning.
(b) About 1 in 40 embankment dams have experienced an accident by piping through the embankment. The most likely to experience an accident are central core earth and rockfill, puddle core earthfill, rockfill and hydraulic fill embankment dams.
(c) Homogeneous dams are more likely to fail than experience an accident, which almost certainly relates to the absence of filters or any zoning to control internal erosion and piping once it begin.
(d) By contrast, while there have been a number of accidents in central core earth and rockfill dams, only one failed (and it had coarse rockfill adjacent to the core with the rock having been placed by derrick). This reflects the fact that most central core earth and rockfill dams have filters or transition zones and high discharge capacity in the downstream rockfill, so while internal erosion and piping may initiate and progress to form a leak, failure is unlikely.
(e) About 1 in 600 embankment dams have failed and 1 in 300 had an accident by piping through the foundation. Dams with narrow cut-offs, such as concrete face earthfill and concrete core wall rockfill, are over-represented in the failures. Accidents are spread over the dam zoning types.
(f) Most failures by internal erosion and piping occur on first filling or in the first 5 years of operation. This may relate to the weaknesses in the dam and foundation being "found out" in this early filling, but there is evidence (Wan and Fell, 2002, 2003) that soils are more erodible when partially saturated than when saturated and this may explain some of the improved performance after 5 years.

It is this history of failure and accidents due to internal erosion and piping which has led to the incorporation of filters and high permeability zones in dams beginning in the 1950s and more particularly in the 1970s. However filters and free draining zones are often costly, so there may be a wish to use simpler designs, such as homogeneous earthfill or zoned earthfill, particularly where the consequences of failure are small. If this is done, those involved should appreciate that there will be a significant chance of failure, particularly for homogeneous dams and if there are other unfavourable circumstances such as dispersive soils, conduits through the dam and/or poor compaction. Foster *et al.* (1998, 2000a, 2000b) give a method for assessing how these factors can affect the historic probabilities in Table 10.2. These are also discussed in Chapter 8.

10.2 TYPES OF EMBANKMENT DAMS, THEIR ADVANTAGES AND LIMITATIONS

10.2.1 The main types of embankment dams and zoning

The main types of embankment dams and zoning are described in Chapter 1. Figures 1.1, 1.2 and 1.3 show the main types of embankment dams. Table 1.1 describes the main zones and their function.

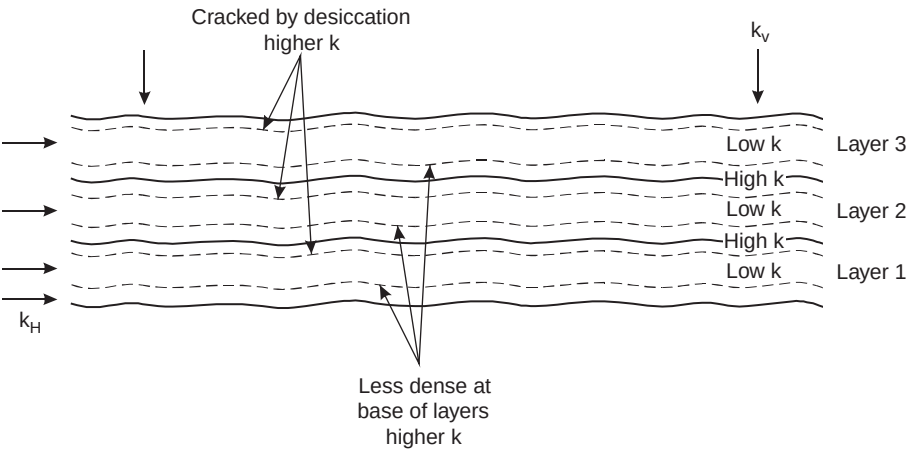


Figure 10.1 Schematic diagram of the effects of compaction layers, and desiccation during construction, on k_H/k_V in compacted earthfill.

10.2.2 The general principles of control of seepage pore pressures and internal erosion and piping

As will be apparent from Section 10.1, much of what differentiates between the likelihood of failure of the different types of dam is the degree of control over internal erosion and piping in the dam and foundation. Second, but still important, is the degree of control over pore pressures in the dam and the foundation so the factor of safety against slope instability can be reliably maintained at adequately low levels.

These are controlled by the permeability of the zones in the dam and the foundation, the ratio of horizontal and vertical permeability k_H/k_V, whether the dam is cracked by differential settlement, desiccation or earthquake, or whether there are flaws or cracks in the earthfill due to poor compaction or desiccation of layers of earthfill during construction.

It is not necessary to have poor compaction to give a high horizontal to vertical permeability ratio, k_H/k_V, which can lead to high pore pressures in the dam. This is shown schematically in Figure 10.1, which shows the normal layering effects of compacted earthfill.

Monitoring of pore pressures in dams shows that k_H/k_V may be anything from 1 to 100.

Figure 10.2 shows schematically the effect of the k_H/k_V ratio in the earthfill on pore pressures in an earthfill dam with a horizontal drain and an earthfill dam with horizontal and vertical drains.

For the earthfill dam with a horizontal filter drain, if k_H/k_V is low (say 1), the seepage through the embankment will flow towards the horizontal drain. Pore pressures in the downstream slope would be low so the factor of safety would be high. If however k_H/k_V is high (say 15), the flow net is quite different, and some seepage

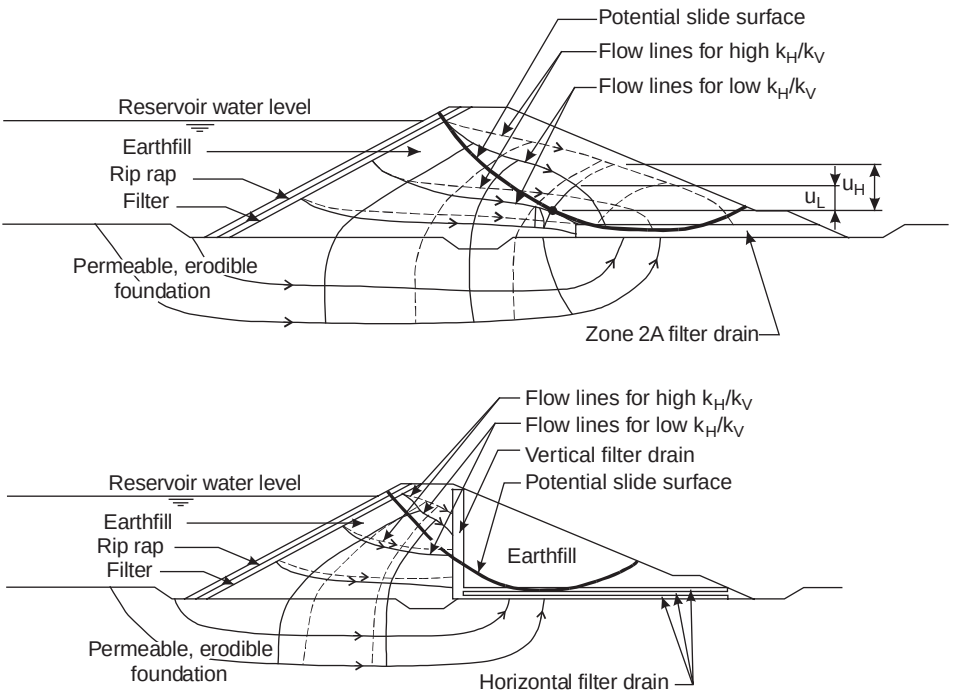


Figure 10.2 Effect of horizontal to vertical permeability ratio k_H/k_V on pore pressures within an earthfill dam (a) with horizontal filter drain, (b) with horizontal and vertical filter drain.

is likely to emerge on the downstream slope. The pore pressures on a potential slide surface are much higher (U_H compared to U_L), so the factor of safety is lower.

For the earthfill dam with a horizontal filter drain (irrespective of the k_H/k_V ratio), the potential for internal erosion and piping is not properly controlled because for dams constructed of plastic soils or soils with high silt content the potential mode of internal erosion is concentrated flow in cracks and not related to the seepage flow net.

For the earthfill dam with horizontal and vertical filter drains, the flow net is similarly affected by the k_H/k_V ratio, but the implications are far less, because the vertical filter drain intercepts the seepage regardless of k_H/k_V ratio, so the pore pressures affecting slope stability are predictably low.

In this case the potential for internal erosion and piping in the embankment is also controlled by the vertical filter drain.

It is important to recognize that it is not possible to predict k_H/k_V, or simulate the effects leading to high k_H/k_V in the laboratory. The way to make the pore pressures more predictable, and to control internal erosion and piping, is to provide filters/drains for the full height of the dam, such as with an earthfill with horizontal and vertical drains or a central core earth and rockfill dam.

Table 10.3 Consequence Categories of dams (ANCOLD, 2012).

	Severity of Damage and Loss			
Population at Risk	*Minor*	*Medium*	*Major*	*Catastrophic*
<1	Very Low	Low	Significant	High C
≥1 to <10	Significant *(Note 2)*	Significant *(Note 2)*	High C	High B
≥10 to <100	High C	High C	High B	High A
≥100 to <1,000	*(Note 1)*	High B	High A	Extreme
≥1,000		*(Note 1)*	Extreme	Extreme

Note 1: With a PAR in excess of 100, it is unlikely Damage will be minor. Similarly with a PAR in excess of 1,000 it is unlikely Damage will be classified as Medium.
Note 2: Change to "High C" where there is the potential of one or more lives being lost.

The other important matter to recognize is that, as discussed above, regardless of k_H/k_V ratio, if an embankment core is cracked by differential settlement, hydraulic fracture or by desiccation, seepage will flow preferentially in the cracks. This will occur quickly (in hours) when the reservoir water reaches the level of the cracking. The rate is not related to the permeability of the compacted earthfill, only the hydraulics of the crack.

Good embankment design recognizes this and provides filters and drains to cope with the flows from such cracking.

If the foundation of a dam is soil, or erodible (e.g. completely weathered) rock, there is potential for internal erosion and piping in the foundation. Good design requires that the seepage is, so far as practical, allowed to flow into a filter drain placed on the foundation as shown in Figure 10.2, or intercepted by pressure relief wells with screens designed to prevent internal erosion.

10.2.3 Taking account of the likelihood and consequences of failure in selecting the type of embankment

It is apparent from Section 10.1 that some types of embankment are much more likely to fail by internal erosion and piping than others. Logically this should be taken into account during the selection process.

Also, logically, the consequences of failure should also be considered, with more reliable types used where the consequences of failure are high. This can be done within a formal risk framework or, in a less formal way, using the population at risk and severity of damage and losses. Organisations such as ICOLD and ANCOLD have classification schemes for consequences of failure, usually called "Hazard" or "Consequence" classification. For the purposes of this book we will use the ANCOLD (2012) Classification. This is shown in Tables 10.3 and 10.4. Refer to that document for the method for details of how to assess the severity of damage and loss, loss of services and business, social and economic and natural environmental damage.

Table 10.4 Explanation of Severity of Damage and Loss (ANCOLD 2012).

Type	*Minor*	*Medium*	*Major*	*Catastrophic*
(a) Total infrastructure costs				
See Below	<$10M	$10M–$100M	$100M–$1B	>$1B
(b) Impact on owners business				
Importance to the Business	Restrictions needed during dry periods	Restrictions needed during peak days and peak hours	Essential to maintain supply	Dissolution of business/entity
Effect on services provided by the owner	Minor difficulties in replacing services	Reduced services are possible with reasonable restrictions.	Severe restrictions would be applied for at least one year	Services cannot be replaced or cannot get services from another source
Effect on continuing credibility	Some reaction but short lived	Severe widespread reaction	Extreme discontent credibility	Total loss of confidence and
Community reaction and Political Implications	Some reaction but short lived	Severe widespread reaction	Extreme discontent credibility	Total loss of confidence and
Impact on financial viability	Able to absorb in one financial year	Significant with considerable impact in the long term	Severe to crippling in the long term	Bankruptcy
Value of water in the storage	Can be absorbed in one financial year	Loss of income for at least 1 year	Loss of income for more than 1 year	Bankruptcy
Health and social impacts				
Loss of service to the community	<100 persons affected	100 to 1,000 people affected	>1,000 persons affected for greater than one month	>10,000 persons affected for a year or more
Cost of emergency management	<1,000 person days	1,000 to 10,000 person days	>10,000 person days	>100,000 person days
Dislocation of people	<100 person months	100 to 1,000 person months	>1,000 person months	>10,000 person months
Dislocation of businesses	<20 business months	20 to 200 business months	>200 business months and some business failures	Numerous business failures
Employment affected	<100 jobs lost	100 to 1,000 jobs lost	>1,000 jobs lost	>10,000 jobs lost
Loss of heritage	Local	Regional	National facility facility	International
Loss of recreational facility	Local facility	Regional developed facility	National facility	International facility

(*Continued*)

Table 10.4 Continued.

Type	*Minor*	*Medium*	*Major*	*Catastrophic*
(c) Environmental impacts				
Area and Duration of impact	$<1\ km^2$	$<5\ km^2$	$<20\ km^2$	$>20\ km^2$
Duration of impact	<1 year	<5 years	<20 years	>20 years
Stock and Fauna	Uncontrolled discharge from dam would not contaminate water supplies used for stock and fauna	Uncontrolled discharge from dam would contaminate water supplies used for stock and fauna. Health impacts not expected.	Uncontrolled discharge from dam would contaminate water supplies used for stock and fauna with contaminant uptake.	Uncontrolled discharge from dam would contaminate water supplies used for stock and fauna with contaminant uptake and measurable health impacts expected.
Ecosystems	Uncontrolled discharge from dam is not expected to impact on ecosystems. Remediation possible.	Uncontrolled discharge from dam would have short term impacts on ecosystems with natural recovery expected after one wet season. Remediation possible.	Uncontrolled discharge from dam would have significant impacts on ecosystems with natural recovery expected after several wet seasons. Remediation possible.	Uncontrolled discharge from dam would have significant permanent impacts on ecosystems. Remediation involves altered ecosystems.
Rare and endangered fauna and flora	Species exist but minimal damage expected. Recovery within one year	Species exist with losses expected to be recovered over a number of years.	Rare and endangered species will be severely impacted Recovery will take many years.	Endangered species will be lost from the area. Permanent loss of species will occur.

10.2.4 Types of embankment dams, their advantages, limitations and applicability

Table 10.5 summarises the types of embankment dams, their degree of control of internal erosion and piping and pore pressures for slope instability, and suitability in relation to the consequences of failure classification.

This table is for new dams and assumes the dams are well designed and constructed so filters meet no-erosion filter criteria and drainage zones have sufficient capacity to discharge seepage through the dam and the foundation.

There are many existing homogeneous, earthfill with toe drains, zoned earthfill and earthfill with horizontal drain, all being dams which have significant and high consequences of failure locations. Whether these dams are suitably safe depends on how well they were constructed, the geometry and geology of the foundations, their performance, the level of monitoring and surveillance and other detailed factors. These can be assessed as detailed in Chapter 8.

Table 10.5 Types of embankment dams, their degree of control of internal erosion and piping, and pore pressures for stability, and suitability in relation to consequences of failure classification.

Embankment type	*Description of the zones*	*Degree of filter control of internal erosion and piping*	*Degree of control of pore pressures for stability*	*Consequences of failure classifications to which suited – new dams*
Homogeneous earthfill	Zone 1 earthfill	None. Seepage in earthfill and from cracks is likely to emerge on the downstream face regardless of k_H/k_V	Poor. Pore pressures not predictable. Depend on k_H/k_V in earthfill	Very low and low
Earthfill with toe drain	Zone 1 earthfill, Zone 4 rockfill, may have Zone 2A/2B filter drain	Poor. Seepage in earthfill with high k_H/k_V and seepage through cracks is likely to emerge on the downstream face. Poor into rockfill if no filters provided	Poor. Pore pressures not predictable. Depend on k_H/k_V in earthfill	Very low and low
Zoned earthfill	Zone 1 earthfill. Zones 1–3 of borrow pit run alluvial silt/sand/gravel; or weathered and low strength rock, compacted to form silt/sand/gravel	Moderate (poor to good). All seepage will be intercepted by Zones 1–3. Depends on particle size distributions of Zones 1–3 to act as a filter to Zone 1	Good provided Zones 1–3 is much higher permeability than Zone 1	Very low to significant, depending on material particle size distributions and construction control
Earthfill with horizontal drain	Zone 1 earthfill. Zone 2A filter drain	Poor. Seepage through earthfill with high k_H/k_V and seepage through cracks is likely to emerge on the downstream face	Poor. Pore pressures not predictable. Depend on k_H/k_V in earthfill	Very low and low. Significant if no population at risk
Earthfill with vertical and horizontal drain	Zone 1 earthfill, Zones 2A, 2B filter drain	Very good to good. Seepage through earthfill and cracks is intercepted by the vertical drain, and provided the drains are designed as filter and have sufficient discharge capacity	Good, provided the filter drains have sufficient capacity to discharge seepage from through the dam and the foundation	Significant to extreme
Earth and rockfill, central core	Zone 1 earthfill, Zones 2A, 2B filters, Zones 3A and 3B rockfill	Very good. Seepage through earthfill and cracks is intercepted by the filters and discharged in the rockfill	Very good, provided the rockfill is free draining	Significant to extreme. Likely to be too complicated and costly for dams less than about 20 m high
Earth and rockfill, sloping upstream core	Zone 1 earthfill, Zones 2A, 2B filters. Zones 3A and 3B rockfill. Earthfill quantity potentially reduced compared to central core earth and rockfill	Very good. Seepage through earthfill and cracks is intercepted by the filters and discharged in the rockfill	Very good, provided the rockfill is free draining	Significant to extreme. Likely to be too complicated and costly for dams less than about 20 m high. Suitable for staged construction
Concrete face rockfill	Zones 3A and 3B rockfill, with upstream Zones 2D and 2E to limit leakage in the event the face slab leaks	Very good. Provided Zones 2D and 2E have particle size distributions to act as filters if they satisfy the requirements of Table 10.6, and be internally unstable	Very good, provided the face slab Zones 2D and 2E are effective, and the rockfill is free draining	Significant to extreme. The cost of plinth and foundation preparation makes CFRD unsuitable for dams less than about 30 m high. Suited for staged construction. Usually not suited for other than rock foundations

10.3 ZONING OF EMBANKMENT DAMS AND TYPICAL CONSTRUCTION MATERIALS

10.3.1 General principles

The following discussion describes "typical" zoning and construction materials for the most common types of embankment dams. Each dam should be designed to satisfy the particular topographic and foundation conditions at the site and to use available construction materials, so there really are no "typical" or "standard" designs. To highlight this, examples are given of the various types of dams. Many of these are drawn from Australian practice as detailed in the ANCOLD (Australian National committee on Large Dams) Bulletins and from the authors' own experience. Figures 1.1 and 1.2 show typical cross sections for the most common types of embankment dams. Table 1.1 describes the zoning numbering system used in Figures 1.1, 1.2 and 1.3 (and throughout this book) and the function of the zones. Table 1.2 describes terms used relating to foundation treatment.

Table 10.6 describes typical construction materials used for the different zones in embankment dams. It is emphasised that good dam engineering involves use of the materials available at the site rather than to look for materials with a preconceived ideas about the material properties needed. However this rule is not followed in the search for Zones 2A and 2B filters where invariably one might seek materials satisfying carefully specified particle size grading limits and dense hard durable materials off-site.

10.3.2 Examples of embankment designs

10.3.2.1 *Zoned earthfill dams*

Figures 10.3 and 10.4 show examples of zoned earthfill design. Both were designed for low or very low consequence of failure classification sites. The Tahmoor dam site was underlain by sandstone and there were limited sources of non-dispersive earthfill. The sandstone rock breaks down to silty sand when compacted in thin layers, yielding an acceptable transition filter material. The design (by the first author) would be improved if Zone 2 was taken to the top of Zone 1 to provide filter protection under flood conditions.

The Blair Athol Dam was built over alluvium, and over an existing dam, which had been built without conventional design or construction control. A cutoff trench was excavated to the weathered sandstone and siltstone foundation. Zoned earthfill construction was considered acceptable with the use of a transition zone of weathered basalt rock compacted in 150 mm thick layers. The "earthfill" was sandstone from mine overburden compacted in 150 mm thick layers.

10.3.2.2 *Earthfill dams with horizontal and vertical drains*

The mine runoff water dam at Drayton Mine (Figure 10.5) is constructed on a foundation of interbedded siltstone and sandstone. It is a low consequence of failure site, allowing the adoption of a design without vertical filter drain. Zone 3 was compacted mine overburden (mostly sandstone), which results in a low permeability earth/rockfill

Table 10.6 Embankment dam typical construction materials.

Zone	*Description*	*Construction material*
1	Earthfill	Clay, sandy clay, clayey sand, silty sand, possibly with some gravel. Usually minimum 15% passing 0.075 mm, preferably more. Note that weathered rock such as siltstone, shale and sandstone can sometimes be compacted in thin layers to give sufficiently fine material
2A	Fine filter	Sand or gravelly sand, with less than 5% (preferably less than 2%) fines passing 0.075 mm. Fines should be non plastic. Manufactured by crushing, washing, screening and recombining sand-gravel deposits and/or quarried rock
2B	Coarse filter	Gravelly sand or sandy gravel, manufactured as for Zone 2A. Zones 2A and 2B are required to be dense, hard durable aggregates with similar requirements to that specified for concrete aggregates. They are designed to strict particle size grading limits to act as filters
2C	Upstream filter and filter under rip rap	Sand gravel/gravelly sand, well graded, e.g. 100% passing 75 mm, not greater than 7% passing 0.075 mm fines non plastic. Usually obtained as crusher run or gravel pit run with a minimum of washing, screening and regarding. Sometimes relaxed durability and filter design requirements compared to Zones 2A and 2B
2D	Fine cushion layer	Silty sandy gravel well graded, preferably with 2–12% passing 0.075 mm to reduce permeability. Obtained by crushing and screening rock or naturally occurring gravels or as crusher run. Larger particles up to 200 mm are allowed by some authorities, but segregation and internal instability is likely to result
2E	Coarse cushion layer	Fine rockfill placed in 500 mm layers to result in a well graded sand/gravel/cobbles mix which satisfies filter grading requirements compared to Zone 2D
3A	Rockfill	Quarry run rockfill, possibly with oversize removed in quarry or on dam. Preferably dense, strong, free draining after compaction, but lesser properties is often accepted. Compacted in 0.5–1 m layers with maximum particle size equal to compacted layer thickness
3B	Coarse rockfill	Quarry run rockfill. Preferably dense, strong, free draining after compaction, but lesser properties is often accepted. Compacted in 1.5–2.0 m layers with maximum particle size equal to compacted layer thickness
4	Rip rap	Selected dense durable rockfill sized to prevent erosion by wave action. In earth and rockfill dams often constructed by sorting larger rocks from adjacent 3A and 3B Zones. In earthfill dams either selected rockfill or a wider zone of quarry run rockfill may be used

with some degree of internal erosion and piping control to Zone 1. A horizontal drain was provided to collect seepage through the foundation in a controlled manner.

The Ash Pond Dam for Loy Yang Power Station (Figure 10.6) is an off river storage founded on Quaternary sediments overlying Tertiary Coal Measures.

The sediments include widely interspersed beds of over-consolidated fissured silty and sandy clays, and clayey and silty sands. The Coal Measures consist of inferior coal and coal seams separated by inter-seam sediments composed of sandy clays, silts and

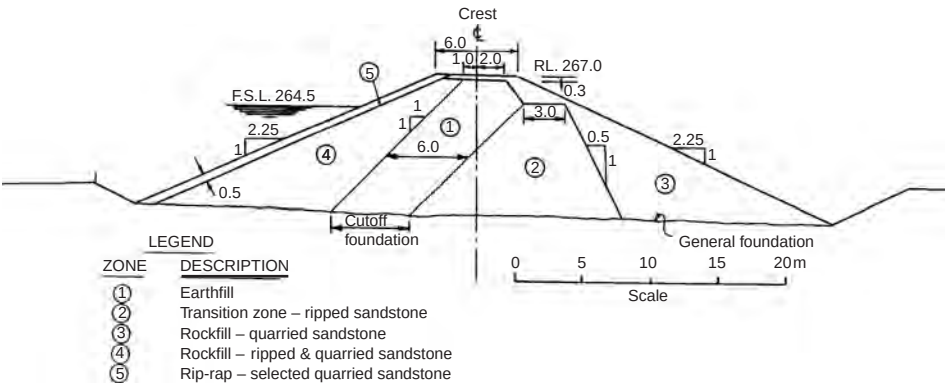


Figure 10.3 Mine water supply Tahmoor Dam (courtesy of Coffey Geosciences).

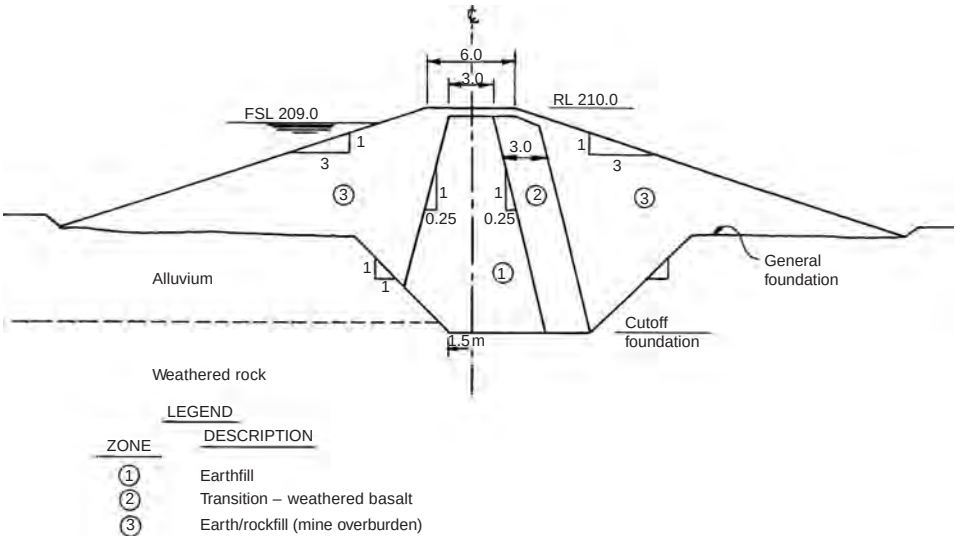


Figure 10.4 Water storage dam, Blair Athol Mine (courtesy of Coffey Geosciences).

silty sands which form a series of aquifers. Increased piezometric levels within these aquifers following filling of the ash storage, required the installation of a series of relief wells at the downstream toe of the dam. The dam has a vertical chimney drain and horizontal drain to control seepage through the dam and its foundation. Internal sand drains were provided to control construction pore pressures.

Plashett Dam (Figure 10.7) is founded on Coal Measure rocks including sandstone, siltstone and coal. The design incorporates a cutoff through alluvial soils in the foundation, a vertical drain and horizontal drainage provided by strips of filter material. (The authors do not favour such strips on erodible foundations as it

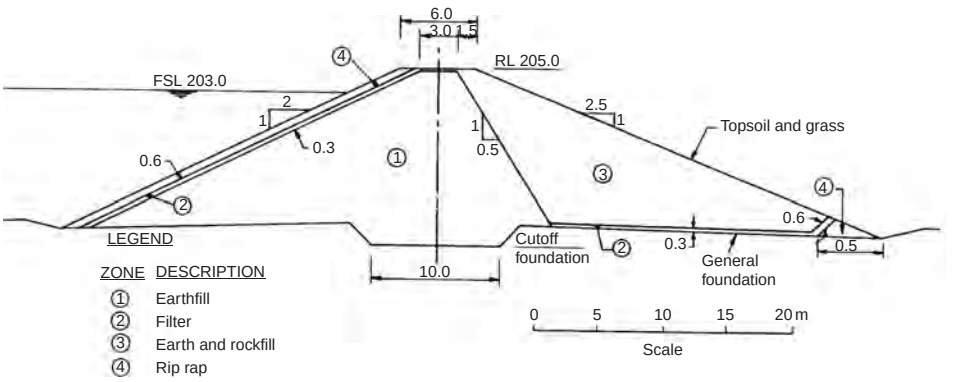


Figure 10.5 Mine runoff water dam, Drayton Mine (courtesy of Coffey Geosciences).

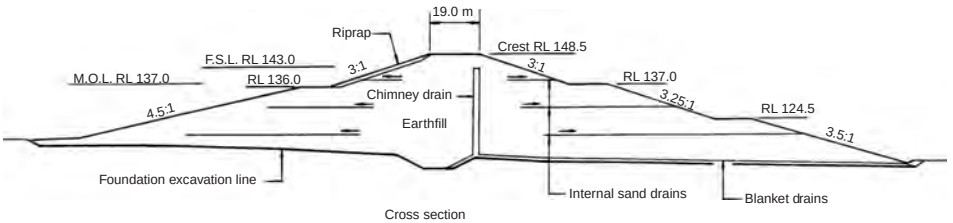


Figure 10.6 Ash Pond Dam, Loy Yang Power Station (courtesy State Electricity Commission of Victoria).

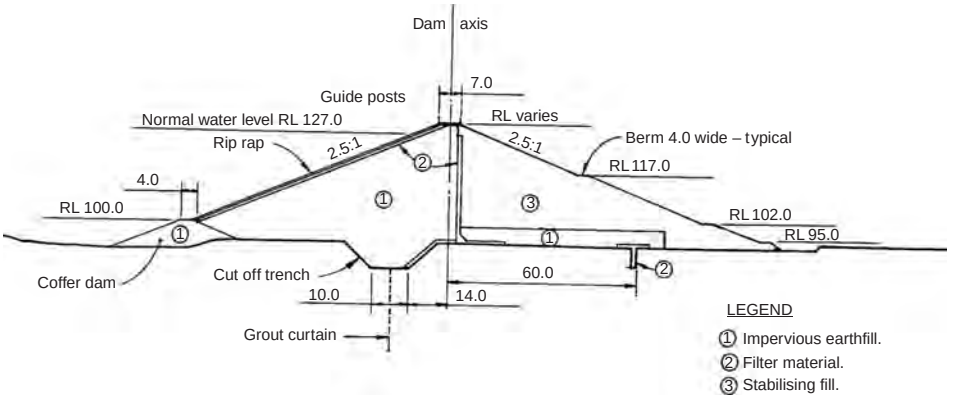
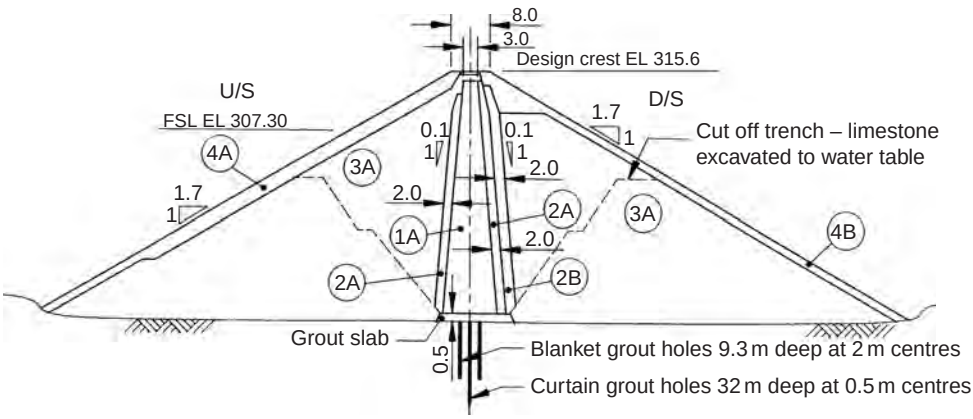


Figure 10.7 Plashett Dam (ANCOLD, 1990).

leaves gaps in the control of internal erosion and piping in the foundation. In this case the drains and cutoff extend to rock so the problem is not so great). A vertical drain is provided through the alluvium to intercept seepage which bypasses the cutoff.



Zone	Description
1A	Clay core
2A	Fine filter
2B	Coarse filter

Zone	Description
3A	Rolled rockfill
4A	Upstream rip rap
4B	Downstream rip rap

Figure 10.8 Bjelke-Petersen Dam (courtesy of Water Resources Commission of Queensland).

10.3.2.3 *Central core earth and rockfill dams*

The Bjelke-Petersen Dam (Figure 10.8) is largely founded on a very complex sequence of andesitic volcanic rock and limestone, which have been intensely sheared and folded. These sequences alternate with metamorphosed phyllite, with steeply dipping fault zones occurring at the contacts between sequences. The limestone exhibited solution channels and sink holes, often infilled with stiff, high plasticity clay. These were less frequent and less permeable below the water table.

The cutoff trench was excavated through the limestone to the water table and a reinforced concrete grout slab formed. A grout curtain, with close hole spacing, was adopted. The core was kept narrow to minimize the cost of excavation and foundation treatment. McMahon (1986) gives more details of the project. See also Chapter 3, Section 3.7.2.

The Lungga Dam (Figure 10.9), in the Solomon Islands, was to have been constructed on 60 m of alluvial sand and gravel. Authors Fell and MacGregor were involved in the investigation and design.

A partially penetrating diaphragm wall was to assist in reducing seepage flow in the foundation, but substantial seepage was still expected. The available rockfill was relatively low permeability, so a substantial horizontal drain and protective filters were incorporated into the design to cope with the very large expected under seepage.

Blue Rock Dam (Figure 10.10) is a fairly conventional central core earth and rockfill dam, constructed on a foundation of mainly Silurian mudstones with thin interbedded sandstones. The rockfill was zoned to accommodate poorer quality rock

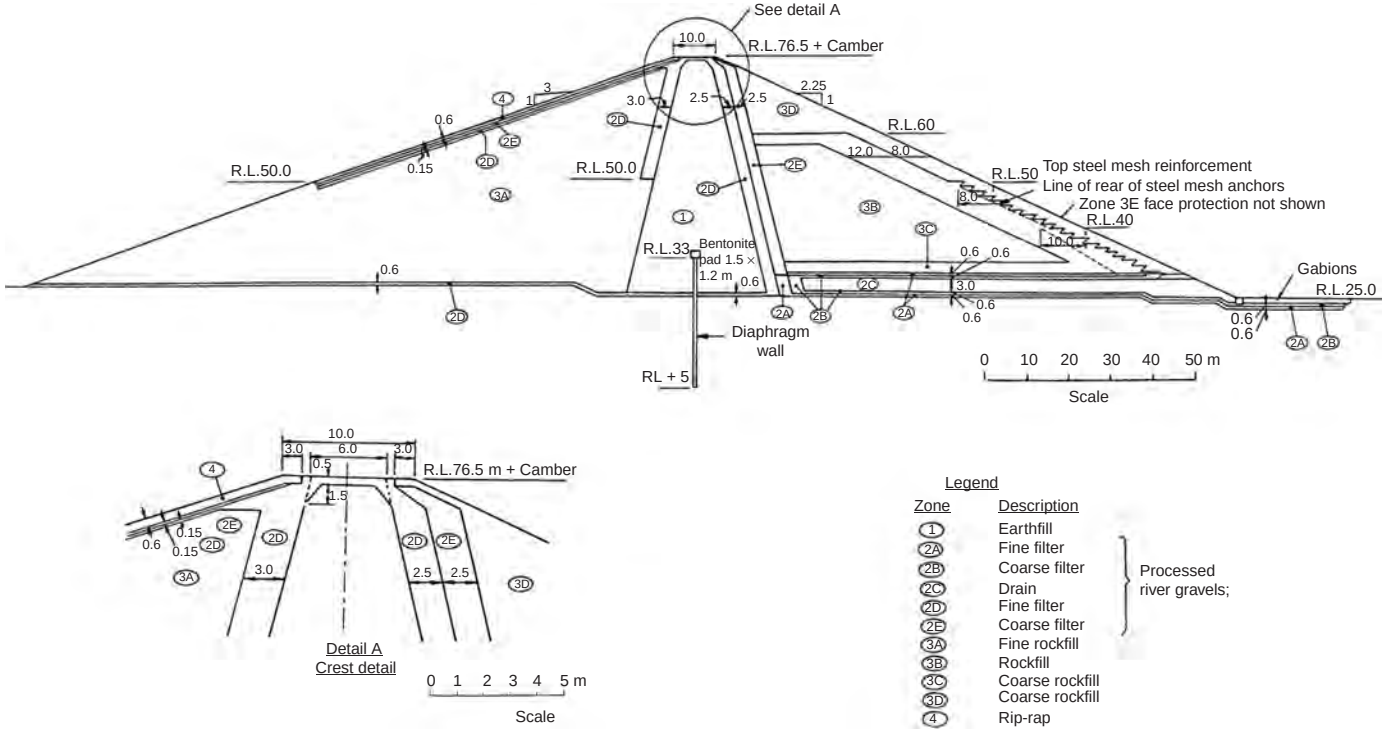


Figure 10.9 Proposed Lungga Dam (courtesy of Coffey Partners International).

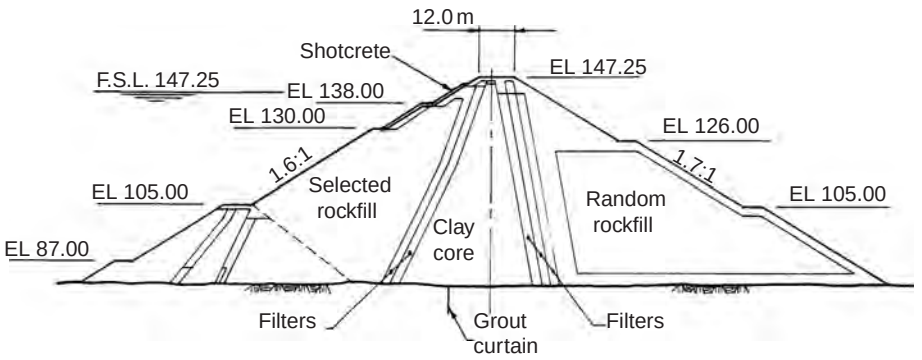


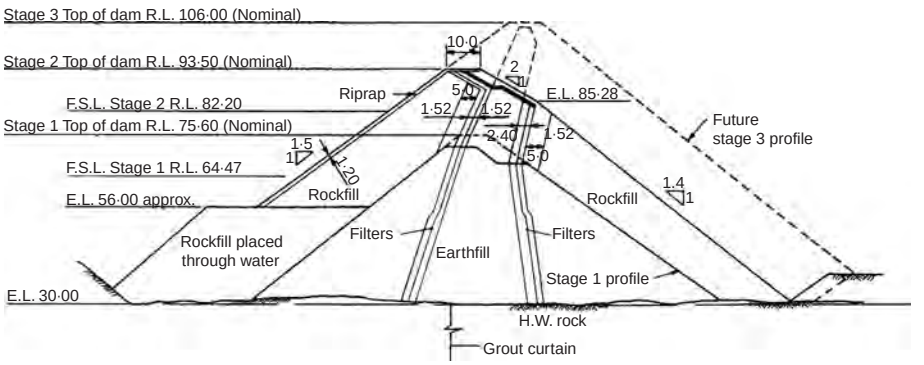
Figure 10.10 Blue Rock Dam (ANCOLD, 1990).

in the random rockfill zone. An unusual feature of the embankment is the use of reinforced shotcrete on the upstream face for wave protection, as the available rock was of inadequate size and quality. Blue Rock design would be improved if the filters were taken higher to control internal erosion and piping under flood conditions higher than expected by the designers.

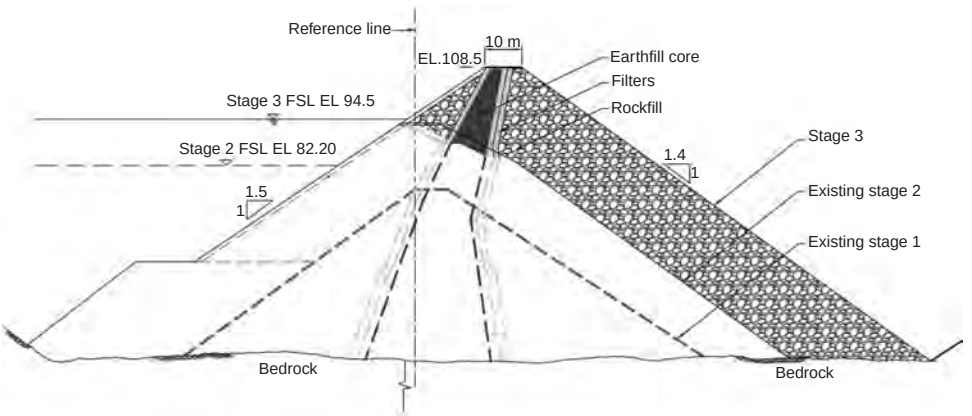
Hinze Dam (Figure 10.11a and b) is a central core earth and rockfill dam which is founded on greywacke, greenstone and chert. It is unusual (for a central core dam) in that it has been designed to be constructed in three stages. This leads to a modified form of geometry, and necessitates placing some rockfill under water. The construction of the third stage of Hinze Dam included raise of the filters to the Stage 3 dam crest level (Hunter and Toose, 2009). Authors Foster, Bell and Fell were involved in Stage 3 investigations and design.

Thomson Dam is 166 m high with a saddle dam on the right abutment. Figures 10.12 and 10.13 show sections through the dam and saddle dam. Authors Bell and Stapledon were involved in investigations and design.

During the design of Thomson Dam, two existing landslides, one of which was identified as a potential massive slide, were identified on the right abutment of the main dam. Several smaller potential slides were also defined under both the main and the saddle dam. Each of the two dams is a rockfill dam with a central earthfill core. The cores are flanked by processed chimney filters, by a fine and a coarse filter on the downstream side, by a fine filter on the upstream side below 7 m below the crest and by a coarse filter on the upstream side in the top 11 m of the dam. The main dam has a partial processed two-filter blanket under its downstream shoulder, except in the top 100 m of the right abutment where the filter blanket was extended under the whole of the downstream shoulder; a full width filter blanket was also placed below the upstream shoulder in the top 80 m of the right abutment; the saddle dam has a full filter blanket under its downstream shoulder. Both dams have riprap zones for wave protection on their upstream faces. Both dams have multi-row grout curtains up to 80 m deep in places. The grout curtain extends beyond the left end of the main dam and right end of the saddle dam and between the dams to form a continuous seepage barrier.



Stage 2 cross section



Stage 3 cross section

Figure 10.11 Hinze Dam (ANCOLD, 1990 and Hunter and Toose, 2009).

A stabilizing fill up to 50 m high was placed upstream of the dam and saddle dam. This is discussed in Section 2.10.3.5 of Chapter 2. The sources of the various zones were:

Zone 1	"Impervious" cores.	Decomposed granite slope wash and residual soil from borrow pits.
Zones 3A, 3C and 4	All rockfill except a small amount of initial main dam stabilizing fill.	Sandstones and siltstones from quarry on left bank. Initial placement in stabilizing fill came from sedimentary rock right bank quarry.
Zone 3B	Upstream face wave protection (riprap).	Fresh granitic rock from right bank quarry.
Zones 2A and 2B	Filters – process materials	Mainly indurated sandstones from quarry on right bank.

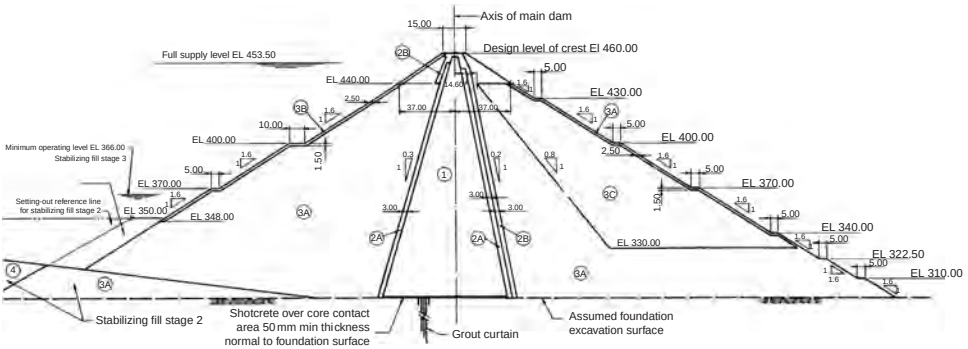


Figure 10.12 Thomson Dam – maximum section of the main dam (Courtesy of Melbourne Water).

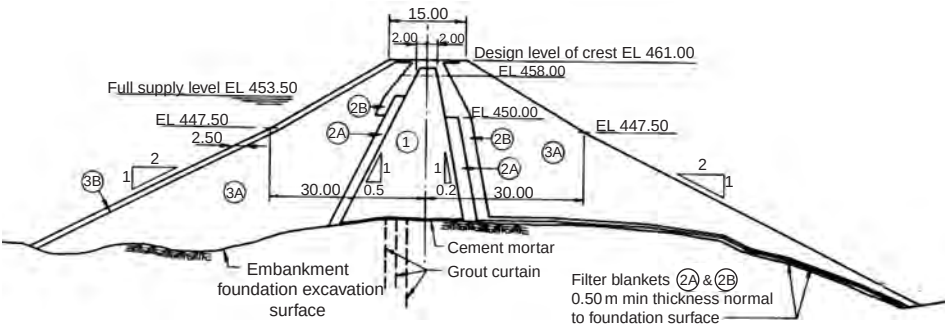


Figure 10.13 Thomson Dam – section through the saddle dam (Courtesy of Melbourne Water).

10.3.2.4 *Sloping upstream core earth and rockfill dam*

An example of a sloping upstream core dam is given in Figure 19.25. The section has been designed to facilitate staged construction. Such a section layout can also be an advantage if there is a lot of lower grade rockfill available as it can, by suitable zoning, be placed as a weighting zone within the downstream shoulder. This section type is particularly suitable for dams that hold reservoirs that are full most of the time, for example, a hydro dam, but the drawdown condition must be carefully checked.

10.3.2.5 *Concrete face rockfill dams*

Several examples of concrete face rockfill dams (CFRD) are given in Chapter 15.

10.4 SELECTION OF EMBANKMENT TYPE

The selection of the type of dam embankment to be used at a particular site is affected by many factors, some of which are outlined below. The dam engineer's task is to consider these factors and adopt a suitable design. The overriding consideration in most cases will be to construct an adequately safe structure for the lowest total cost.

Hence, preparation of alternative designs and estimates of cost for those alternatives will be a normal part of the design procedure. Usually the most economic design will be that which uses a construction materials source close to the dam, without excessive modification from the "borrow pit run" or "quarry run" material.

The following outlines some of the factors involved and their effect on embankment type.

10.4.1 Availability of construction materials

10.4.1.1 *Earthfill*

Clearly the availability of suitable earthfill within economic haul distance is critical in the selection of the embankment type. If there is no earthfill available – for example in an area underlain by sandstone, which weathers to give only a shallow cover of sandy soil, it will normally be appropriate to construct a rockfill dam with concrete (or other) impervious membrane, or a concrete gravity dam.

The uniformity of the available earthfill will also influence design and the method of construction. If the borrow areas produce two different types of earthfill, the earthfill may be zoned into two parts, e.g. for an earthfill dam with vertical and horizontal drains, the earthfill with lower proportion of fines and more variable properties would be best placed downstream of the vertical drain. Alternatively the earthfill zone may be separated into Zones 1a and 1b with the coarser soil in Zone 1b adjacent the Zone 2A filter.

Alluvial clayey soils are often more variable than residual soils derived from weathering of underlying rocks and, hence, it may be necessary to provide additional zoning as described above. Alternatively, the soils may be mixed by borrowing from a vertical face with a shovel and truck operation (rather than use of scrapers). Blending of soils on the dam embankment is generally avoided as it leads to increased cost and difficulty in quality control.

If cobbles and boulders are present in clayey soil deposits, these will have to be removed prior to compaction either by passing the earthfill through a "grizzly" or by grader or hand labour on the embankment. This is necessary to prevent the oversize particles affecting compaction.

Relatively permeable soils can be used for earthfill in many dam projects. The permeability of most dam foundations is between 1 Lugeon and 10 Lugeons (10^{-7} m/sec to 10^{-6} m/sec) so for most dams an earthfill core with permeability, say 10^{-9} m/sec, seepage through the foundations will far exceed that through the dam. Even if silty sand is used for the earthfill, a permeability of 10^{-6} m/sec should be achieved, i.e. not higher than the foundation permeability and from a seepage viewpoint such a high permeability soil would be acceptable for many dams. Weathered or even relatively unweathered siltstone and sandstone with a clayey matrix, may break down sufficiently when compacted in thin layers to achieve a satisfactory core material. This will usually require field trials to observe the actual properties. MacKenzie and McDonald (1985) and ANCOLD (1985) describe trials and actual performance of compacted siltstone and sandstone, during construction of Mangrove Creek concrete face rockfill dam.

10.4.1.2 *Rockfill*

Over the last 40 years the unit rate for construction of rockfill has reduced compared to that for earthfill and as usually less rockfill is required (because the side slopes can be steeper), the availability of rock which can be quarried to yield free draining rockfill often leads to economic dam design.

Most igneous and many metamorphic rocks, e.g. granodorite, diorite, granite, basalt, rhyolite, andesite, marble, greywacke, quartzite and even low grade metamorphics such as some indurated siltstones and sandstones will, when fresh, yield free draining rockfill.

Some metamorphic rocks, e.g. phyllite, schist, gneiss and slate, may break down under compaction to yield a poorly draining rockfill even though it may be dense with a high modulus. The amount of breakdown depends upon the degree to which foliation or cleavage is developed in the rocks.

Most highly weathered, and many moderately weathered igneous and metamorphic rocks, will not yield totally free draining rockfill.

The spacing of the bedding and joint planes influences the size and grading of rockfill obtained from a quarry. Blasting may be varied to yield the required sized product, but this is not always practicable.

Thick beds of sandstone within a sequence of thinner bedded siltstone and sandstone are likely to yield oversize rock, which would require either secondary breaking in the quarry or sorting and disposal on the embankment.

Often a substantial amount of the rockfill for a dam will come from "required excavations", i.e. from the spillway, foundation for the dam, inlet and outlet works etc. This is in principle desirable as the effective cost of the rockfill could be only the cost of placement (and any additional haulage costs). However, the rock quality from these excavations may not be ideal (due to rock type, weathering, method of excavation) necessitating changes to the embankment zoning to accommodate the material. Also the timing of production of rock from required excavations may not be ideal from the viewpoint of dam construction and in the event may not be as scheduled. Some flexibility in zoning is desirable to allow for such circumstances. Such flexibility is also useful to allow for different quality of rock being obtained from required excavations from that expected at the time of designing the embankment.

Most sedimentary rocks, e.g. sandstone, siltstone, shale and mudstone generally tend to break down under compaction, even when fresh, and yield poorly draining rockfill. In these circumstances it may be necessary to incorporate zones of free draining rockfill to ensure the embankment rockfill as a whole is capable of remaining free draining, e.g. Figures 10.9 and 10.10. In some cases it has been found that the surface breakdown on a layer of weaker rockfill can block any near-vertical drainage paths within the rockfill zone.

The use of so-called "random" rockfill zones in a dam section also facilitates use of rock, which does not yield free draining rockfill. Care is needed in the use of random zones that they do not block off drainage paths. Thus it may be necessary to underlie a random zone with a selected free-draining zone of quality rockfill.

10.4.1.3 *Filters and filter drains*

A source of high quality sand and gravel is necessary for construction of filters. It may be necessary to obtain these materials from off-site sources a long way from the dam site (50 km is not unusual), despite the haulage costs.

Filter aggregates may be obtained from alluvial sand and gravel deposits, or more commonly from quarries. Generally suitable aggregates are of igneous and, less commonly, metamorphic origin. It is unusual to manufacture filters from sedimentary rocks, as these rocks are usually not sufficiently durable and often have poor shape (as measured by flakiness index), but such was done successfully at Thomson Dam from indurated sandstone.

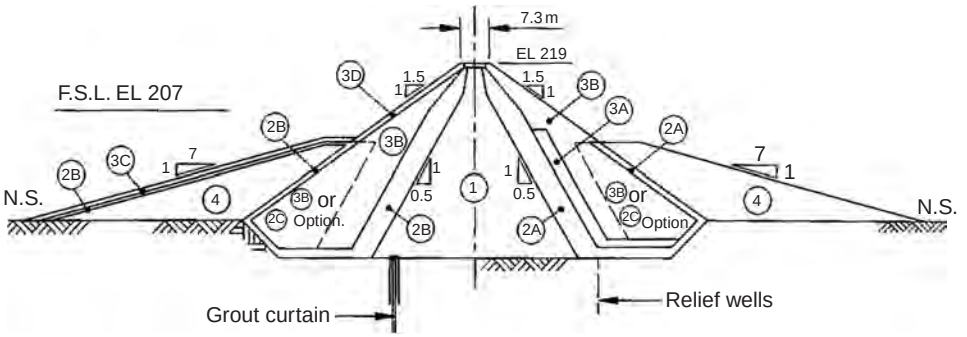
For large dams it is usually necessary to establish a separate crushing and screening plant for manufacture of filter and concrete aggregates. It is sometimes necessary to let a separate early contract to begin the manufacture and stockpiling of aggregates, as their production rate controls dam construction progress.

Where sources of filter aggregates are far from the dam and for this or other reasons filters are expensive, the width of filter zones may be reduced by using equipment such as spreader boxes (see Chapter 9).

10.4.2 Foundation conditions

The strength, permeability and compressibility of the dam foundation have a major influence on the embankment type, e.g.:

- A soil foundation will have a relatively low strength, which may determine the embankment stability and will require relatively flat embankment slopes. Such a formation is likely to favour construction of earthfill dams, e.g. earthfill with horizontal and vertical drains, rather than earth and rockfill;
- A permeable soil foundation will be susceptible to leakage and internal erosion, requiring construction of some form of cutoff and a filter drain under the downstream slope of the dam (see Section 10.5).
- A strong low permeability rock foundation is suited to any type of dam construction, but may favour construction of a concrete face rockfill, a concrete gravity or, in particular cases, a concrete arch dam.
- In earthquake zone areas, the presence of loose to medium dense saturated sandy soils in the foundation will be important, as liquefaction may occur during earthquakes. This may necessitate removal or densification of the sandy soil and/or the provision of weighting berms.
- Dams on karst limestone foundations are a special case, where extensive grouting and other work may be needed to limit leakage to acceptable levels. Such situations favour adoption of a design, which allows for grouting to continue during embankment construction or after it is completed. This may tend to favour concrete face rockfill or earth and rockfill with a sloping upstream core. Alternatively a concrete gravity dam may be adopted so grouting can be carried out from the drainage gallery.
- In some sedimentary rocks, particularly interbedded weak claystone and mudstone and strong sandstone, which have been subject to folding and/or faulting, bedding



Zone	Description
1	Selected core material
2A	Washed gravelly sand
2B	Pit run gravelly sand
2C	Free draining gravelly sand

Zone	Description
3A	Rockfill – Quarry fines
3B	Rockfill
3C	Rip rap
3D	Rip rap (See spec.)
4	Miscellaneous fill

Figure 10.14 Maroon Dam (courtesy of Water Resources Commission of Queensland. Note: Drawing is not to scale).

plane shears may exist, resulting in low effective friction angles (Hutchinson, 1988, Casinader, 1982). In these cases, flat slopes may be required on the embankment, favouring earthfill with vertical and horizontal drains, or earth and rockfill with random rockfill zones. Figure 10.14 shows the cross section for Maroon Dam, where such conditions required very flat slopes of 7H:1V for the weighting berm.

– In some areas, often but not always tropical climate, the rock is deeply weathered and sometimes with a lateritic profile which may lead to a high permeability, soil strength foundation, favouring embankments with flatter slopes and good under drainage, e.g. earthfill with vertical and horizontal drain.
– Embankments constructed on deep soils, e.g. deep alluvium in or adjacent to the river bed, may be subject to a large amount of settlement, leading to differential movement and cracking. In such dams it is particularly important to provide good filters to control seepage and prevent internal erosion.

10.4.3 Climate

It is difficult (and in many cases impossible) to construct earthfill embankments during wet weather, or in freezing temperatures. This is particularly critical when the rain is relatively continuous without high evaporation (and not so critical when the rain is in short storms, followed by hot sunshine).

In these circumstances, it is often advantageous to adopt concrete face rockfill or sloping upstream core construction, so that the rockfill can continue to be placed in the wet weather and the face slab or core constructed when the weather is favourable.

In very arid areas there may be a shortage of water for construction, thus favouring concrete face rockfill rather than earthfill.

10.4.4 Topography and relation to other structures

The selection of embankment type and overall economics of a project is determined with consideration of all components of the project, i.e. embankment, spillway, river diversion outlet works etc. These components are interrelated, e.g.:

- The diversion tunnel will be longer for an earthfill dam (with relatively flat side slopes) than for a rockfill dam.
- The spillway may generate rockfill (and possibly earthfill and random fill) for use in the embankments. The size of spillway can be varied by storing more floodwater in the reservoir, necessitating a higher embankment but smaller spillway so the optimisation of total project cost may influence embankment zoning and size.
- It is common practice in Australia to allow large floods during construction to pass over the embankment, rather than providing a larger diversion tunnel and coffer dam. This may require incorporation of some rockfill in the downstream toe of the dam with steel mesh reinforcement or the use of a long gentle slope in the partially completed downstream shoulder (see Chapter 13).

The topography of the site, i.e. valley cross section or slope, curve of the river in plan or the presence of "saddles" in the abutments, can have a significant effect on embankment selection, e.g.:

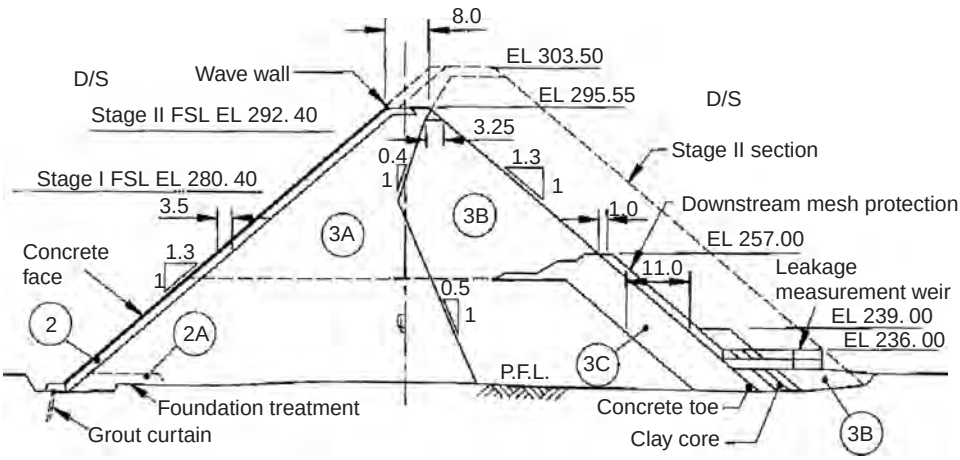
- In narrow steep sided valleys there is restricted room for construction vehicles and haul roads, favouring embankments with simple zoning, e.g. concrete face rockfill or concrete gravity dam
- The curve of the river in plan, and changes in valley cross section, may favour adoption of an upstream sloping core rather than central core (or vice versa) to reduce the quantities of earthfill.
- Local changes in slope of the abutments may lead to differential settlement and cracking, necessitating more extensive filter drains, or favouring concrete face rockfill construction.

10.4.5 Saddle dam

Directly related to the question of topography is a group of dams called saddle dams. These are secondary dams which close low points in the reservoir rim.

Experience suggests that saddle dams have been often treated as "distant" to the main dam in projects and less attention has been applied to investigations and design. A particular issue that can cause significant problems to the designer of a saddle dam is the dam's foundations. Tops of ridges often present different geological conditions with greater depths of weathering than one would expect to find in a valley. As a result failure of the saddle dam may erode well into the foundation.

At first glance, the loss of a saddle dam might seem minor insofar as downstream consequences or even in loss in storage. The experience of a fuse plug failure at Silver



Zone	Description
2	Face material
2A	Filter material

Zone	Description
3A	Rockfill-up to 600 mm dia.
3B	Rockfill-up to 900 mm dia.
3C	As per 3B with no more than 5% passing through a 100 mm aperture

Figure 10.15 Boondooma Dam (courtesy Water Resources Commission of Queensland).

Lake Dam in the U.S. (FERC, 2011) in which erosion went a long way below the plug's foundation should serve as a warning to an owner assuming the loss of a saddle dam would have little impact on his business. This warning also applies to those estimating the dam breach characteristics for dam break studies for evaluating the potential consequences of saddle dams where they are founded on deep erodible foundations.

A saddle dam must be treated in the same way and to the same detail as a main dam. An experienced engineering geologist should be asked to investigate any significant saddle dam site in a dam project.

10.4.6 Staged construction

It is often economic to construct a dam in two or more stages, e.g.:

- In water supply, irrigation or hydropower projects, demand in the early years can often be met with a lower dam and smaller storage.
- In mine tailings dams, the storage required increases progressively as the tailings are deposited in the dam.

If staging is planned, this favours adoption of concrete face rockfill, earth and rockfill with sloping upstream core, or possibly earthfill with vertical (or sloping) drain and horizontal drain. Figures 10.11, 10.15 and 19.24, show examples of dams designed for staged construction.

10.4.7 Time for construction

The time available for construction may influence the selection of dam type, particularly if considered in relation to other factors such as the climate, e.g. in a climate of well defined wet and dry seasons it may be practicable to construct an earth and rockfill or earthfill dam, but only over two dry seasons. A concrete face rockfill dam may be constructed in lesser time by continuing to place rockfill in the wet season.

Foundation treatment and zoning details may also be influenced by the time available for construction. For example, if constructing a dam on a permeable soil foundation, cutoff may be achieved by a cutoff wall at the upstream toe rather than a rolled earth cutoff under the central core, so that the cutoff wall can be constructed at the same time as the rest of the embankment. As discussed in Section 10.8 such an arrangement would require careful detailing of the connection between the top of the cutoff wall and the embankment due to the high gradients across the wall (Figure 10.30b).

The speed of construction is also a significant factor as this affects the contractors and owners overhead costs. This has also favoured roller compacted concrete gravity dams in recent times.

10.5 GENERAL REQUIREMENTS AND METHODS OF CONTROL OF SEEPAGE AND INTERNAL EROSION AND PIPING IN EMBANKMENT DAMS AND THEIR FOUNDATIONS

The objective in designing a new dam, or when assessing an existing dam, is to be satisfied that the dam has adequate measures to control seepage and potential internal erosion in the dam and its foundation so that:

(a) Pore pressures in the dam and the foundation are such that there is an adequate margin of safety against slope instability.
(b) Internal erosion, which might progress to form a pipe and breach the dam, will not occur. This applies to piping in the embankment, in the soil or rock foundation, from the embankment to the foundation and the foundation into the downstream zones of the embankment.

These objects are achieved by zoning of the dam, provision of filters and high seepage discharge capacity zones and other embankment design and foundation treatment measures. Figure 10.16 shows some of the measures which are used.

The design features are:

A: Low permeability "core" of the dam, to limit seepage through the dam.
B: Chimney filter drain, which intercepts seepage through the dam and controls internal erosion of the core and pore pressures in the dam, if the downstream zone C is low permeability.
C: Downstream zone, which maintains stability and may if permeable compared to the core and filter, control pore pressures.

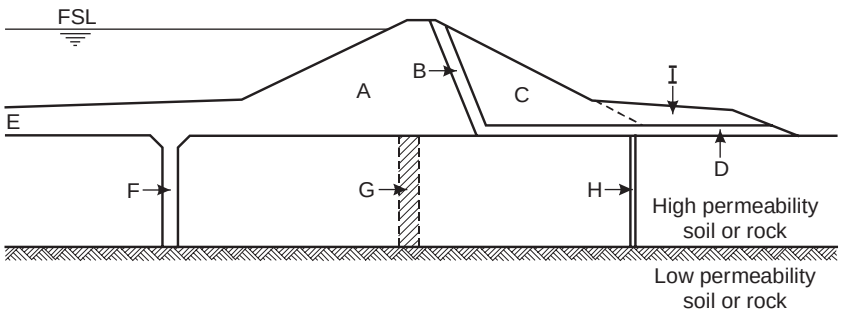


Figure 10.16 Design features for controlling seepage and erosion through and beneath earth dams on highly permeable foundations A, low permeability core; B, chimney filter drain; C, downstream zones; D, horizontal filter drain; E, upstream low permeability blanket; F, slurry trench; G, grouted zone; H, relief wells; I, weighting berm.

D: Horizontal filter drain – which controls exit gradients, erosion of the foundation and pore pressures (but may actually increase the amount of seepage because the gradients are increased).

E: Upstream low permeability blanket which increases the seepage path, reduces seepage, and seepage exit gradients.

F: Slurry trench (or other types of cutoff) which will reduce (or almost stop if fully penetrating to a low permeability base) seepage and seepage exit gradients (usually in soil or extremely weathered rock).

G: Grouting to reduce seepage and seepage exit gradients. Grouting is usually in rock, since grouting in soil is usually costly and ineffective.

H: Pressure relief wells – control exit gradients where confined aquifers in the foundation reduce the effectiveness of the horizontal drain.

I: Weighting berm – which improves downstream slope stability and overcomes the potential for liquefaction or "blow-up" or heave of the foundation (with H).

Of course, not all these features are needed for every dam. For example, a central core earth and rockfill dam on a strong, non erodible rock foundation will usually have grouting, but will not need a horizontal filter or drain.

10.6 SOME PARTICULAR FEATURES OF ROCK AND SOIL FOUNDATIONS WHICH AFFECT SEEPAGE AND INTERNAL EROSION CONTROL

Seepage occurs through all embankment dams and their foundations. The permeability of most compacted earthfill core materials is less than 10^{-8} or 10^{-9} m/sec. By comparison virtually all rock foundations have rock mass permeability greater than 1 lugeon (1 lugeon = 1 litre/second/metre of drill hole at a pressure of 1000 kPa) or approx. 10^{-7} m/sec, and most rocks have a greater permeability than 5 lugeons (approx. 5×10^{-7} m/sec). Foundations of alluvial or deeply weathered and lateritised rock may

Table 10.7 Example of the relationship of rock mass permeabilities to weathering, joint infill and stress relief effects.

Zone	*Depth to base of zone (m) Range (Avg.)*	*Permeability (m/sec × 10^{-7})*	
		Avg.	*Range*
Weathered rock with clay infilled joints	0–12 (8)	1.5	0.1–54
Weathered rock (limonite stained and coated joints)	19–27 (22)	8.0	0.1–27
Rock in stress relief zones	20–36 (28)	108.0	4.8–>200
Average rock mass zone (fresh rock)	–	2.6	0.1–10

have mass permeability as high as 10^{-3}–10^{-5} m/sec. As a result most of the seepage is through the foundation, not the embankment and significant rates of seepage may occur.

The following details some particular features of rock and soil foundations which influence seepage in the foundation and the design of seepage control measures.

(a) *Rock foundation.* As discussed in detail in Chapters 3 and 4, the permeability and seepage in rock foundations is controlled by joints, bedding, shears and faults and in some situations other features such as solution cavities.

- The detailed characterisation and understanding of the hydro-geological model is vital to design of foundation grouting, assessing the likely magnitude of the under seepage and erodibility of the foundation and hence the need for filters, drains or other high permeability zones in the dam. It should be noted that clay infilling of joints and stress relief at depth can give marked stratification of permeability's in weathered surfaces.
- Table 10.7 shows the assessed rock mass permeability model for the Ben Lomond Tailings dam site which had steeply dipping meta-sediments in the foundations. Similar effects were present at the Ranger Tailings dam, giving a confined aquifer about 4 metres below the surface, below the clay infilled joints in the upper weathered rock. This necessitated construction of weighting berms to maintain adequate factors of safety against heave or erosion into rockfill in the dam if filters are not provided over the foundations.
- Some rocks, such as basalt, inter-bedded sandstone and shale with stress relief joints, limestone and granite with stress relief effects, are particularly susceptible to having open joints. If these rocks are exposed in the cutoff foundation, the core material may erode into the joints by contact erosion or scour. If the rock around the joints is completely weathered it may erode into the open joints by concentrated leak erosion under seepage in the open joints from the reservoir.

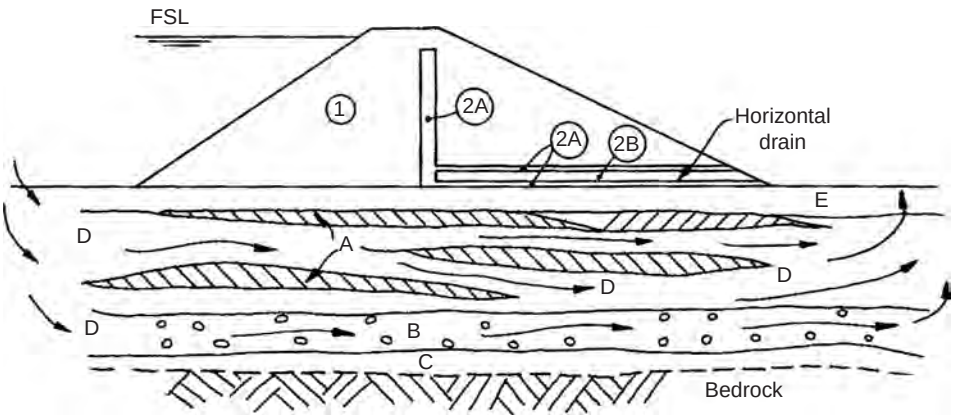


Figure 10.17 Some common features of alluvial foundations.

(b) *Alluvial Soils*. Some features of alluvial foundations are shown schematically in Figure 10.17. They include:

A: Lenses or layers of lower permeability sand, silty sand, or even clayey sand may occur, giving a very much reduced vertical permeability. These may be present in point bar deposits.

B: There is often a coarser gravel, or even boulder/gravel layer at the base of the alluvium (channel lag deposit), reflecting the time when the river was more active. This may be very permeable.

C: The upper part of the rock surface may be permeable because of the presence of open joints, potholing and de-stressing. The surface may also be very irregular.

D: The coarse alluvium – sand/gravel is in itself likely to be layered giving a high permeability ratio k_H/k_V, e.g. point bar deposits.

E: There is often a layer of silty sand/silty sandy gravel on the surface, giving a low permeability, e.g. in flood plain deposits.

In general these features combine to give a much greater horizontal than vertical permeability. Hence while water can enter into the foundation in the large area of exposed alluvium in the reservoir, it may be inhibited from flowing into the horizontal drain. Similar layering can occur in soils of colluvial origin.

Internal erosion may occur in the foundation e.g. by suffusion of internally unstable, gap-graded, sandy gravels and by contact erosion of fine grained soils into adjacent coarse grained soils. As well as the particle size distribution of the soils, the continuity of the layers and whether they persist under the dam and exit in a way in which eroded material can be transported from the foundation is important, but often difficult to assess with confidence.

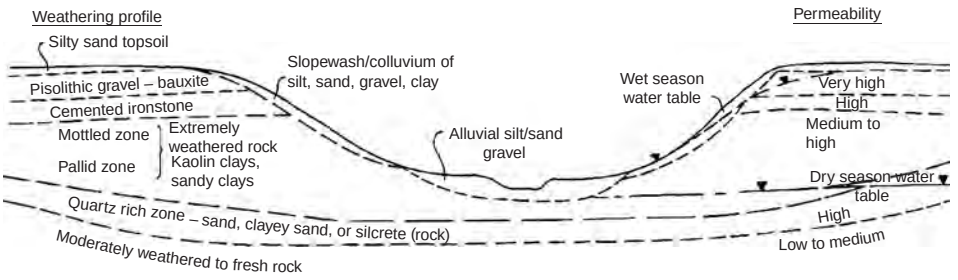


Figure 10.18 Some common features of lateritic foundations.

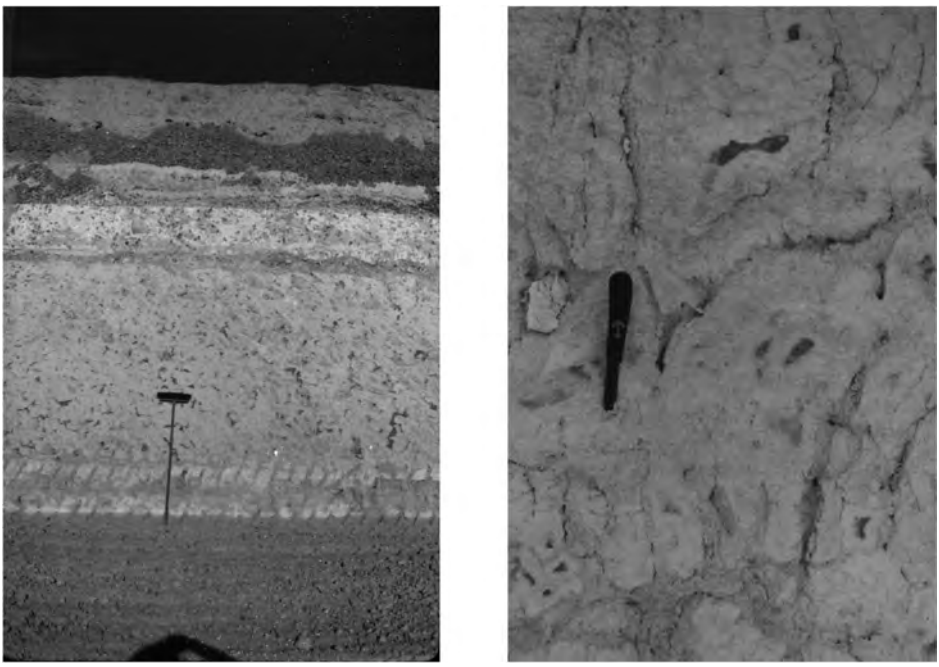
(c) *Lateritised soil and rock profiles*. Figure 10.18 shows some common features of lateritised soil and rock profiles which have developed by weathering of rock. The features include:

- Very high permeability upper zone of pisolithic gravel.
- Variably cemented and permeable ironstone zone.
- Mottled zone and underlying kaolin rich zone. This is commonly medium to high permeability due to relict joints and root holes as shown in Figure 10.19. Many of these features are near vertical and not readily intercepted by vertical site investigation boreholes. The mass permeability of the clay will commonly be of the order of 10^{-6} to 10^{-5} m/sec, i.e. the permeability of fine sand.
- The quartz rich zone overlying weathered bedrock is often a mixture of clean sand and clayey sand of high permeability. In some cases it may be cemented as silcrete, but will still be highly permeable.
- The upper bedrock is often fractured and has moderate permeability.
- The depth of weathering may be up to 30 m, e.g. in the Darling Ranges of Western Australia (Gordon, 1984) and in the subtropical monsoonal climate areas of Weipa, North Queensland and Ranger and Jabiluka, Northern Territory. The weathering profile usually does not follow the topography but rises more gradually in the abutments as shown in Figure 10.18. At Weipa, Ranger and Jabiluka, with low relief, it is the authors' experience that the depth of weathering is related to rock type rather than topography.
- The water table fluctuates markedly between end of wet season and end of dry season, often by as much as 10 m or 20 m.

10.7 DETAILS OF SOME MEASURES FOR PORE PRESSURE AND SEEPAGE FLOW CONTROL

10.7.1 Horizontal and vertical drains in the embankment

Seepage beneath a dam on a permeable soil (or permeable weathered rock) foundation should be allowed to exit in a controlled manner into a horizontal drain. The horizontal



(a) Overall profile, bauxite at top overlying pallid, kaolinite zone.

(b) Close-up showing sand infilled root holes and fissures.

Figure 10.19 Example of sand infilled roots and fissures in a lateritic formation – Weipa, North Queensland.

drain may consist of a single layer of Zone 2A filter, or 3 layers of Zone 2A and 2B as shown in Figure 10.17. In both cases Zone 2A should be designed to act as a filter to control erosion of the soil from the foundation into the drain. The filter design criteria detailed in Chapter 9 should be used.

The 3 layer drain incorporates the layer of high permeability Zone 2B to ensure that the drain has sufficient discharge capacity.

It is good practice (Cedergren, 1972) to design the horizontal drain to have sufficient capacity to discharge the flow entering the drain from the dam foundation and from the vertical drain without the phreatic surface rising into the low permeability fill (see Figure 10.20).

If the horizontal drain has insufficient capacity, the phreatic surface will rise into the downstream low permeability fill as shown in Figure 10.21, reducing the stability of the downstream slope.

Such lack of drain capacity can also occur for dams on permeable rock foundations where earthfill dams such as those shown on Figures 10.20 and 10.21 are used. It can also occur where rockfill with a high percentage of silty fines has been used as the downstream zone.

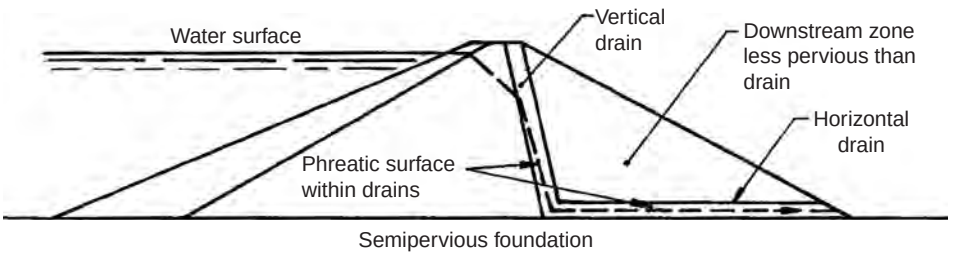


Figure 10.20 Earth dam with internal drain designed to prevent the phreatic surface from rising above the top of the drain.

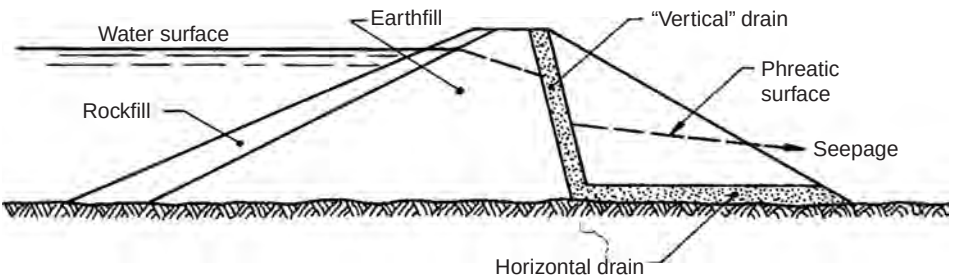


Figure 10.21 Earth dam with internal drains of inadequate discharge capacity.

Cedergren (1972) gives a design method for estimating the discharge capacity of a horizontal drain without pressurization based on:

$$q = \frac{k_1 h^2}{2L_1} \tag{10.1}$$

where k_1 = permeability of the drain material (m/sec)
h = vertical thickness of the drain (m)
L_1 = length of the drain (m) as shown in Figure 10.22b.
q = discharge capacity per metre width of drain (width measured across river) (m^3/m/sec).

The capacity of the vertical drain is seldom a critical issue because the quantity of seepage through the earthfill is small and the vertical drain width is dictated by construction factors. However its capacity should be checked by:

$$q_2 = \frac{k_2 h_2 w}{L_2} \tag{10.2}$$

where k_2 = permeability of vertical drain (m/sec)
h_2, L_2 (m) are as shown in Figure 10.22(a)
w = width of drain (m).

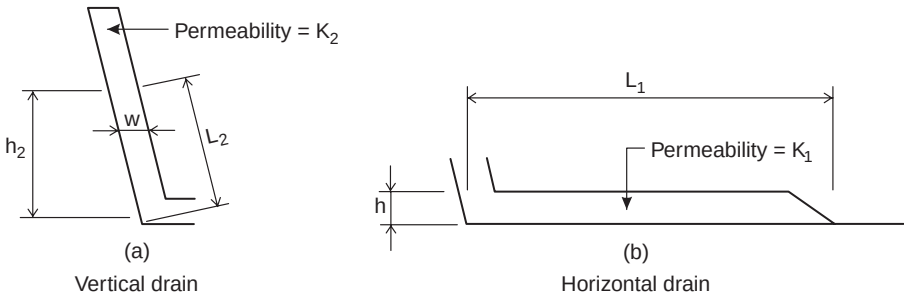


Figure 10.22 Design of drain dimensions for discharge capacity.

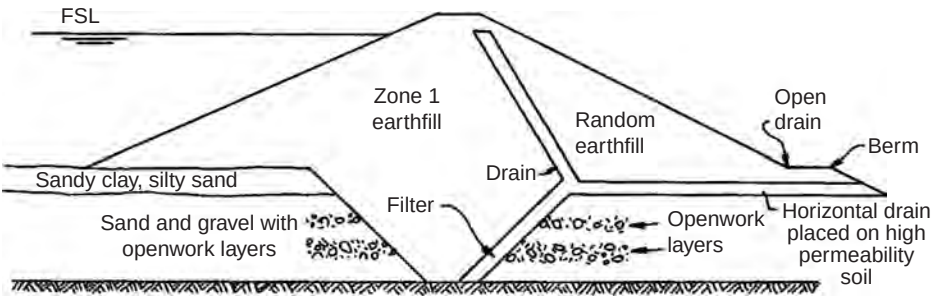


Figure 10.23 Control of erosion of earthfill into permeable foundation using a filter.

If there is any doubt about the capacity, a two (2A, 2B) or three (2A, 2B, 2A) layer filter drain system should be used.

When checking the design of the horizontal drain the following should be noted:

- Water from the abutments will flow towards the lowest part of the drain before flowing out the toe of the dam. Hence the required flow capacity per unit width in this area will be greater than the average flow per unit width under the dam.
- If a three layer filter drain is used, Zone 2B will dominate the discharge capacity.
- Conservative estimates of foundation permeability should be used for the design of the horizontal drain, since failure to provide adequate capacity can lead to failure of the dam, or the requirement for a stabilizing berm if the problem is recognised.
- Similarly, conservative estimates of the horizontal drain permeability should be used, and care taken to ensure the drain is not contaminated by soil from the foundation during construction. Generally it is good practice to provide a three layer drain, so there is a large discharge capacity.
- The downstream toe should be designed to ensure the drain outlet is not blocked by erosion of soil off the embankment or the abutment. It is wise to provide a berm at the toe of the dam above the drain to collect any material eroded off the downstream face of the dam (as shown in Figure 10.23).

- Place the filter drain in contact with the higher permeability strata in the foundation. So, as shown in Figure 10.23, the lower permeability sandy clay-silty sand has been removed.
- Specify that the foundation not be rolled or trafficked by construction equipment which would potentially destroy the natural drainage paths in the top part of the foundation, so reducing the permeability under the filter drain and restricting the flow of seepage into the drain. This is a common problem for existing dams and very high pore pressures have built up beneath the drain in some cases where the foundation was compacted prior to placing the filter.

10.7.2 Treatment of the sides of the cutoff trench

Erosion of the earthfill core into the sides of the cutoff trench may occur where:

- There are open jointed rocks, with a joint opening large enough to allow the soil toerode into the joint e.g. in columnar basalt and other volcanics, pyroclastic rocks, open-jointed sandstone or other rock.
- Coarse gravels in alluvial or colluvial soils are present.
- There are fissures or holes in lateritic soils or weathered rock.

Figure 10.23 shows one way of controlling erosion into a soil foundation. The filter would have to satisfy critical filter criteria (see Chapter 9) in regard to the Zone 1 earth-fill and the "openwork" layer of alluvium. In open jointed rock it is normal to cover the open joints with shotcrete or other concrete, before placing the earthfill. This may also be done in cutoffs into soil and weathered rock.

10.7.3 Prevention of critical seepage gradients and heave of the foundation

If seepage water emerges at too high a gradient from downstream of the toe of a dam on alluvial foundations, critical seepage gradients may be reached resulting in what is termed "heave", "blow-up", "boiling" or "liquefaction."

This results in:

- Loss of shear strength, potentially leading to slope instability;
- Potential for development of backward erosion piping in non-plastic soils, particularly uniform fine or medium sand. This is usually evidenced by the presence of sand boils.

Figure 10.24 shows the area (A–B) which is susceptible.

Figures 10.25 and 11.14 show another situation where heave is likely to occur; where a low permeability layer in the foundation confines the seepage flow.

The mechanism involved is one of the high pore pressures in the foundation leading to low effective stresses, with heave occurring when the effective stress becomes zero.

The factor of safety against this occurring can be calculated in two ways for Point X in Figure 10.24, as either:

$$F_{UT} = \frac{\sigma_v}{u} \tag{10.3}$$

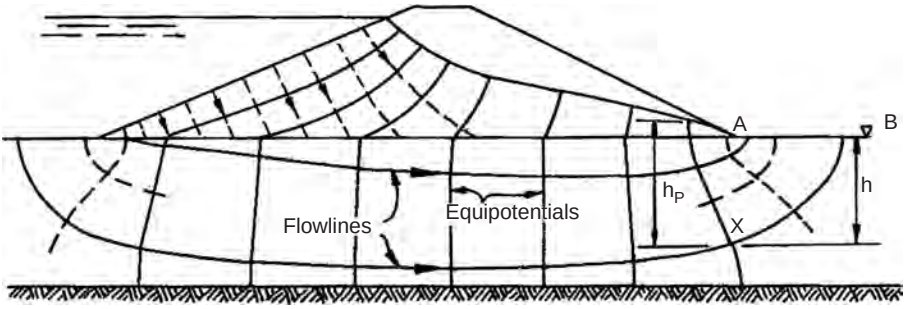


Figure 10.24 Locations downstream of a homogeneous dam where blow-up may occur.

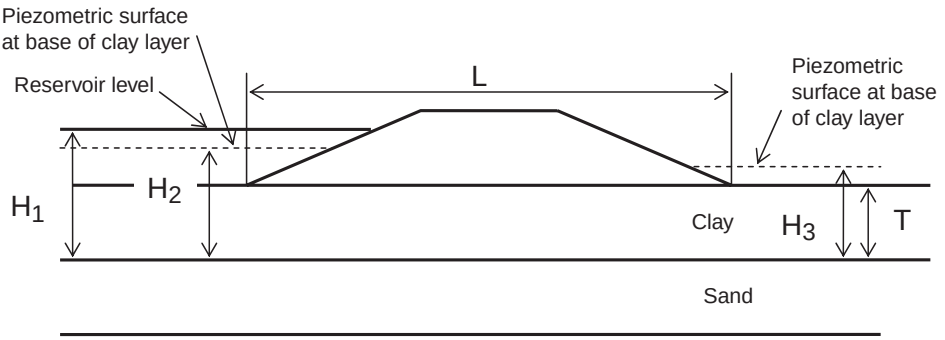


Figure 10.25 Section through embankment or levee founded on alluvial clay overlying sand showing definition of terms.

where σ_v = total vertical stress at any point in the foundation (kN/m^2)
u = pore pressure at the same point (kN/m^2)

or:

$$F_{UT} = \frac{h\gamma_{sat}}{h_p\gamma_w} \tag{10.4}$$

where γ_{sat} = unit weight of saturated foundation soil (kN/m^3)
γ_w = unit weight of water (kN/m^3)
h_p = piezometric head (m).

The factor of safety against heave calculated in this manner should be compatible with the consequences of failure and degree of certainty of predicting pressures. However it should be at least 1.5, with pore pressures calculated conservatively (Cedergren, 1972).

The alternative method for estimating the factor of safety is to consider the gradient of the flow net. If the gradient approaches the critical gradient, liquefaction and heave can be expected to occur.

$$F_{UC} = \frac{\text{critical gradient}}{\text{actual gradient}} \tag{10.5}$$

For vertically up seepage, the critical gradient i_{cr} can be calculated from (Terzaghi 1960)

$$i_{cr} = \frac{G - 1}{1 + e} \tag{10.6}$$

where G = soil particle density (specific gravity) in tonnes/m^3
e = void ratio

or from

$$i_{cr} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} \tag{10.7}$$

The authors' preference is to use the zero effective stress, F_{UT} definition, because this allows a better understanding of the ways in which the factor of safety can be improved, and it is easier to measure the pore pressures at critical locations such as the base of the confining layer than local gradients.

Where there is a layer of cohesive or other low permeability soil overlying the soil which may be subject to backward erosion, such as in Figure 10.25, the most critical gradient usually occurs from the base of the lower permeability layer to the ground surface.

The factor of safety on the critical gradient calculated in this manner should also be compatible with the consequences of failure and degree of certainty of predicting pressures. USACE (2000) suggest that the factor of safety should be at least 1.7, midway between pressure relief wells. The authors believe that it is more difficult to predict seepage gradients than to predict pore pressures as required for the heave calculation and would seek to have a high factor of safety, say 3.

USACE (1956), Wolff (2002) and Shannon and Wilson (1995) record that sand boils often occur at average gradients, defined as residual head at toe divided by thickness of the top strata (=(H3 – T)/(T) in Figure 10.25), between 0.5 to 0.8 compared to the theoretical critical gradient of 0.85 for soils with the properties commonly found along the Mississippi River. Shannon and Wilson (1995) record cases of sand boils with average gradients at the toe of 0.12 to 0.84. It appears from an initial review of these cases that most may have not been repeated sand boil activity so represent first time boils.

However it should be noted that if the clay strata in Figure 10.25 are penetrated by excavations, or holes formed by decayed roots backward erosion piping may occur without a heave condition being present. Also, if the base of a clay confining layer is fissured, this may reduce the effective thickness of the confining layer due to the transmission of pore pressures from the underlying high permeability soils into the fissured soils if they are more permeable.

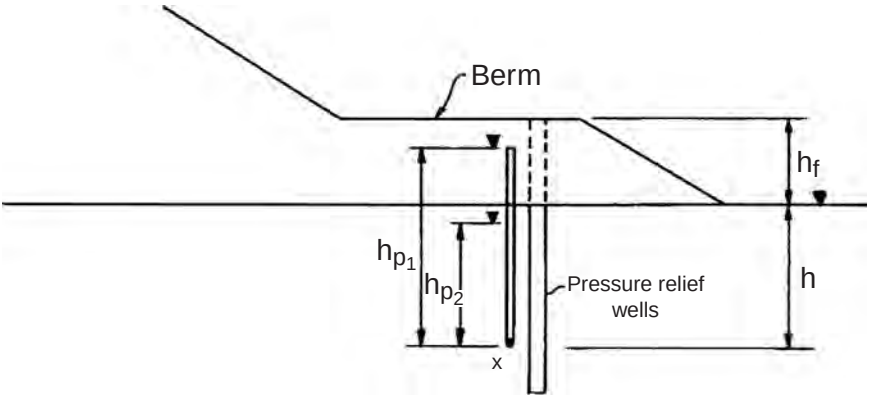


Figure 10.26 Control of "blow-out" or liquefaction by pressure relief wells and weighting berm.

In the event that pore pressures are calculated or known from piezometers to be too high, the factor of safety can be increased by adding a weighting berm (see Figure 10.26) and/or by providing pressure relief wells. The weighting berm is effective by adding to the vertical stress; the pressure relief drains by reducing the pore pressure and provides a working platform to install and maintain the pressure relief wells.

If the original pore pressure at X was hp_1, the original factor of safety would have been:

$$F_{UT} = \frac{\gamma_{sat} h}{\gamma_w hp_1} \tag{10.8}$$

If a berm of height h_f and unit weight γ_f were constructed, then

$$F_{UB} = \frac{\gamma_{sat} h + \gamma_f h_f}{\gamma_w hp_1} \tag{10.9}$$

If no berm were provided but instead pressure relief wells were constructed, resulting in a reduction of pore pressure to $\gamma_w hp_2$, then

$$F_{UW} = \frac{\gamma_{sat} h}{\gamma_w hp_2} \tag{10.10}$$

If both the berm and pressure relief wells were constructed

$$F_{UWB} = \frac{\gamma_{sat} h + \gamma_f h_f}{\gamma_w hp_2} \tag{10.11}$$

The following should be noted:

– It is difficult to predict pore pressures near the toe of a dam or levee, because they are greatly affected by local variations in permeability (see for example Figures 11.14 and 11.15), assumptions on the ratio of horizontal to vertical permeability,

and the permeability of the strata overlying the sand. These factors should be considered in carrying out the finite element analyses of seepage and the sensitivity of the pore pressures to assumptions checked. It is necessary to be conservative and to provide piezometers to check the assumptions.

– The berm will be most effective if it is free draining, at least for the situation shown in Figure 10.24.

Pressure relief wells are designed to relieve artesian pressures and are usually constructed to be self-draining at the original ground surface or below if they drain into drain pipes installed below ground surface level.

– The pressure relief wells will provide greatest benefit at each well and least benefit midway between wells. For conservatism the design should be based on the pore pressures midway between wells. Again, piezometers are required to check the effectiveness of the wells.

10.7.4 Design of pressure relief wells

USACE (2000) provides useful information on the design of pressure relief wells. Some of the important points are:

(a) The wells should penetrate through the permeable strata. Partially penetrating wells may not be effective.
(b) The spacing of wells depends on the depth, stratification and permeability of the foundation soils; distance to the source of seepage; characteristics of the strata overlying the high permeability strata; the degree of pressure relief desired.
(c) Wells are potentially subject to corrosion and bacterial clogging or carbonate encrustation and must be designed so the screens can be cleaned by flushing and/or replaced (i.e. therefore require access to the top of the wells).
(d) The water collected by the wells may have water quality issues and may need to be collected and treated.

Figure 10.27 shows a well design from USACE (2000). They indicate this design has been used widely and has given good service. They make the following points:

(a) *Riser pipe and screen*. The well screen normally extends from just below the top of the permeable stratum to the bottom of the well, with a solid riser pipe installed from the top of the permeable strata to the surface. In zones of very fine sand or silt interbedded with the permeable strata the screen is replaced by unperforated (blank) pipe. The screen and riser are most commonly stainless steel or plastic because they are corrosion resistant. However plastic risers should be considered with caution as they are susceptible to damage.
(b) *Filter*. The filter should be designed according to the methods outlined in Chapter 9. USACE (2000) uses the Sherard and Dunnigan (1989) approach. The filter has to control erosion in the soil surrounding the well, and not erode into the well screen.
(c) *Well installation*. Proper methods of drilling, backfilling and development of relief wells must be used or the well will be of little use. USACE (2000) gives details.

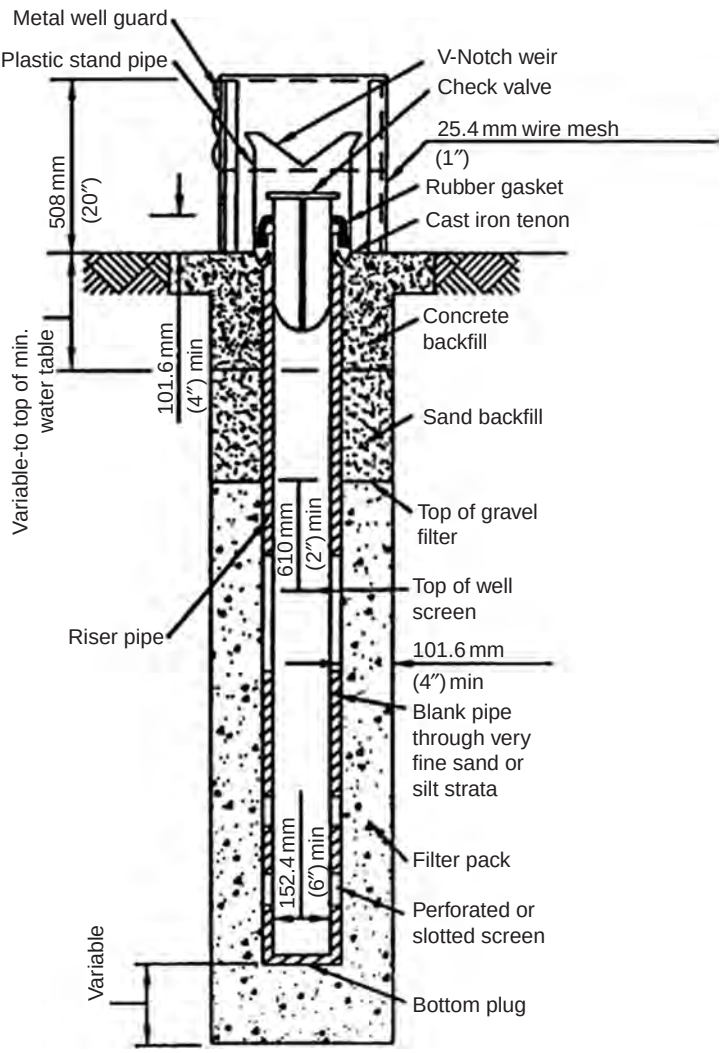


Figure 10.27 Typical USACE relief well (USACE, 2000).

10.8 CONTROL OF FOUNDATION SEEPAGE AND INTERNAL EROSION AND PIPING BY CUTOFFS

10.8.1 General effectiveness of cutoffs

As shown in Figure 10.16, seepage through a permeable foundation can be reduced by constructing a low permeability cutoff through the permeable material. This may consist of:

- Cutoff trench filled with earthfill;
- Slurry trench;

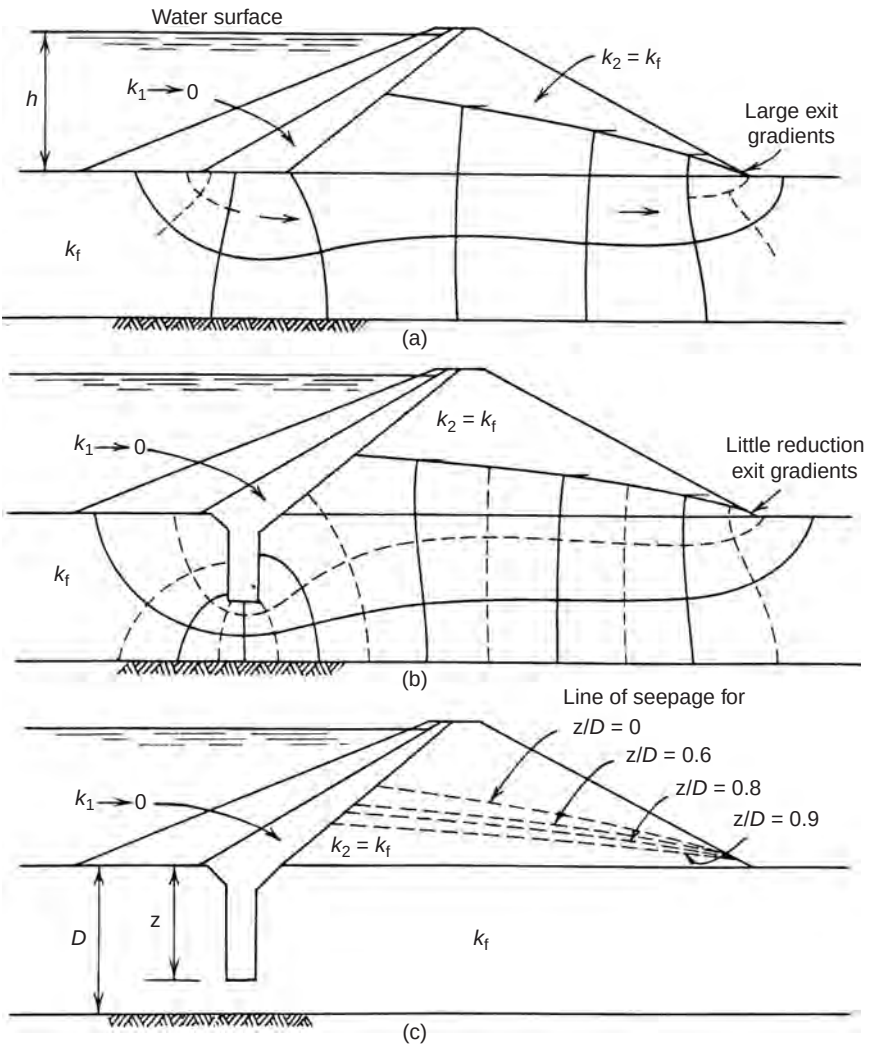


Figure 10.28 Effect of partial cutoff on position of line of seepage: (a) Flownets, $z/D = 0$; (b) Flownet, $z/D = 0.6$; (c) Position of line of seepage to various values of z/D (Cedergren, 1972).

- Diaphragm wall using rigid or plastic concrete;
- Contiguous or intersecting bored piles;
- Sheet pile wall;
- Grout curtain.

Such cutoffs will have varying degrees of effectiveness of controlling seepage and internal erosion depending on their permeability and the depth to which they are taken. This is illustrated in Figures 10.28 and 10.29 which are reproduced from Cedergren (1972).

Figure 10.28 shows the effect on the line of seepage in the downstream shell of an embankment and the effect on exit gradients of cutoffs constructed to varying

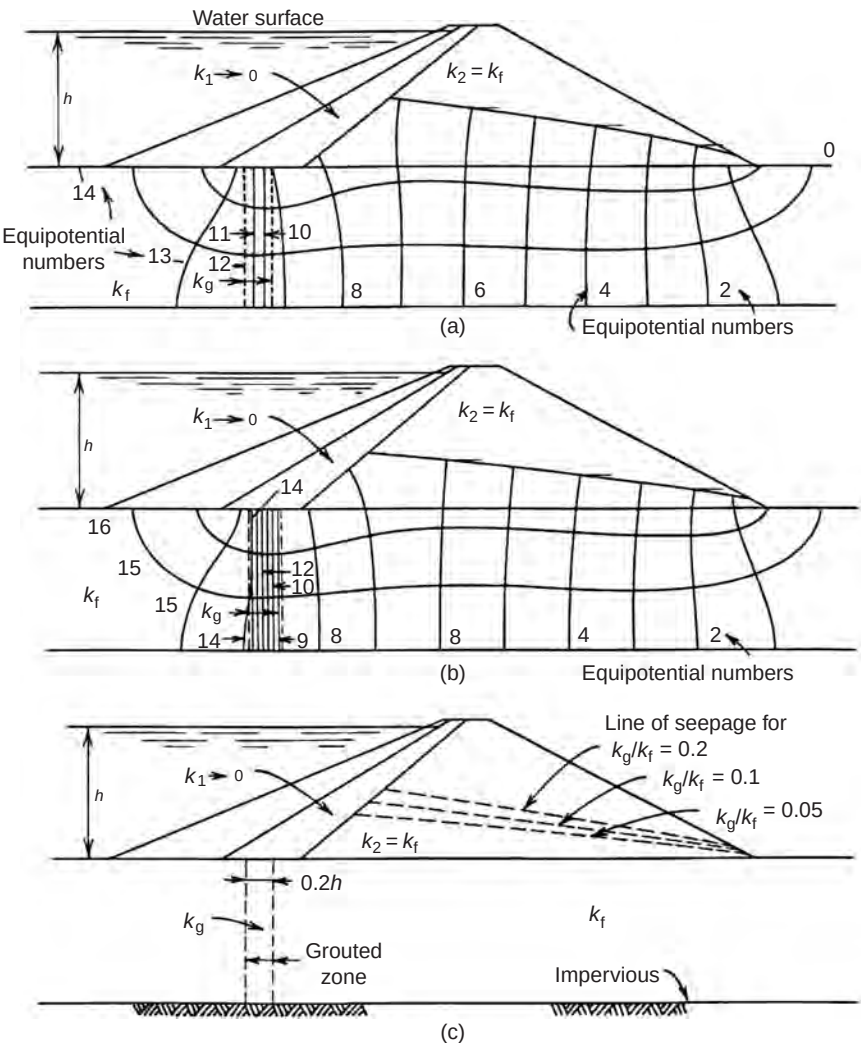


Figure 10.29 Effect of grouted cutoffs on position of line of seepage: (a) Flownet, $k_g = 0.2k_f$; (b) Flownet, $k_g = 0.1k_f$; (c) Position of line of seepage for various values of k_g/k_f (Cedergren, 1972).

proportions of the total depth of a permeable foundation (note the example assumes the foundation and downstream fill zone have the same permeability).

It can be seen that there is not a significant improvement even with a cutoff which is 90% penetrating. Only where the cutoff is fully penetrating and properly connected into the low permeability zone will the seepage pressures be controlled. This is often difficult to achieve as will be discussed below.

Figure 10.29 shows the effect of fully penetrating grouted cutoffs on the line of seepage in the downstream zone. Unless the grouted zone (or cutoff constructed

by another method) has permeability much less than the foundation, there is little reduction in downstream pore pressures. This applies to soil and rock foundations.

The effectiveness of partially penetrating cutoffs will depend on layering of lower and higher permeability soils in the foundation. If there are continuous low permeability layers present, partially penetrating cutoffs can be effective. However it must be expected that, unless a cutoff is fully penetrating and of low permeability (say 10 to 100 times less than the permeable foundation), it will have little benefit in the reduction of leakage and exit gradients.

Figure 10.30 shows some important implications of constructing effective slurry trench or similar cutoffs. As shown in Figures 10.30(a) and (b), the seepage through the dam may flow towards the foundation downstream of the cutoff and a filter may be required under the earthfill to prevent erosion. Figure 10.30(c) shows the problem of how far a cutoff should penetrate into the abutments, to prevent seepage flowing around the ends.

Foundation cutoffs control internal erosion by breaking the continuity of potential pathways for piping in the foundation. Cutoff walls have also been installed through existing embankments in order to reduce the potential for internal erosion through the embankment or internal erosion at the embankment-foundation interface. Some issues for the design of cutoffs to address internal erosion include:

- The ends of the cutoff should terminate within non-erodible materials. This is because the hydraulic gradient will locally be high around the ends of the cutoff wall once it is installed. However this is not always practical to achieve, especially on the sides. In these cases, additional control measures such as filter protection may be required to protect potential seepage pathways around the ends of the wall.
- Ideally the cutoff wall material should have good erosion resistance so that if a defect such as cracks or gaps develops in the cutoff wall, then the concentrated seepage through the wall defect will not make the defect larger and the flows will be limited.
- For cutoff walls installed from the base of the dam foundation, the connection between the top of the cutoff wall and the core zone requires special detailing due to locally high gradients and potential for differential settlement of the fill around the wall.
- Careful consideration of the risks of installing a cutoff wall through existing embankment dams and that measures are in place to manage these risks. Examples include the possibility of intercepting filter zones, hydraulic fracturing of core zones by trench fluids and trench collapse.

10.8.2 Cutoff trench

When the depth of permeable soil is relatively small, an effective way of providing a cutoff is to excavate a trench through the permeable layer and backfill with Zone 1 earthfill as shown in Figures 10.4 and 10.23. If there are continuous relatively low permeability layers within the soil the cutoff trench may be stopped at such a layer rather than penetrating to rock provided the potential for internal erosion below the cutoff wall is low.

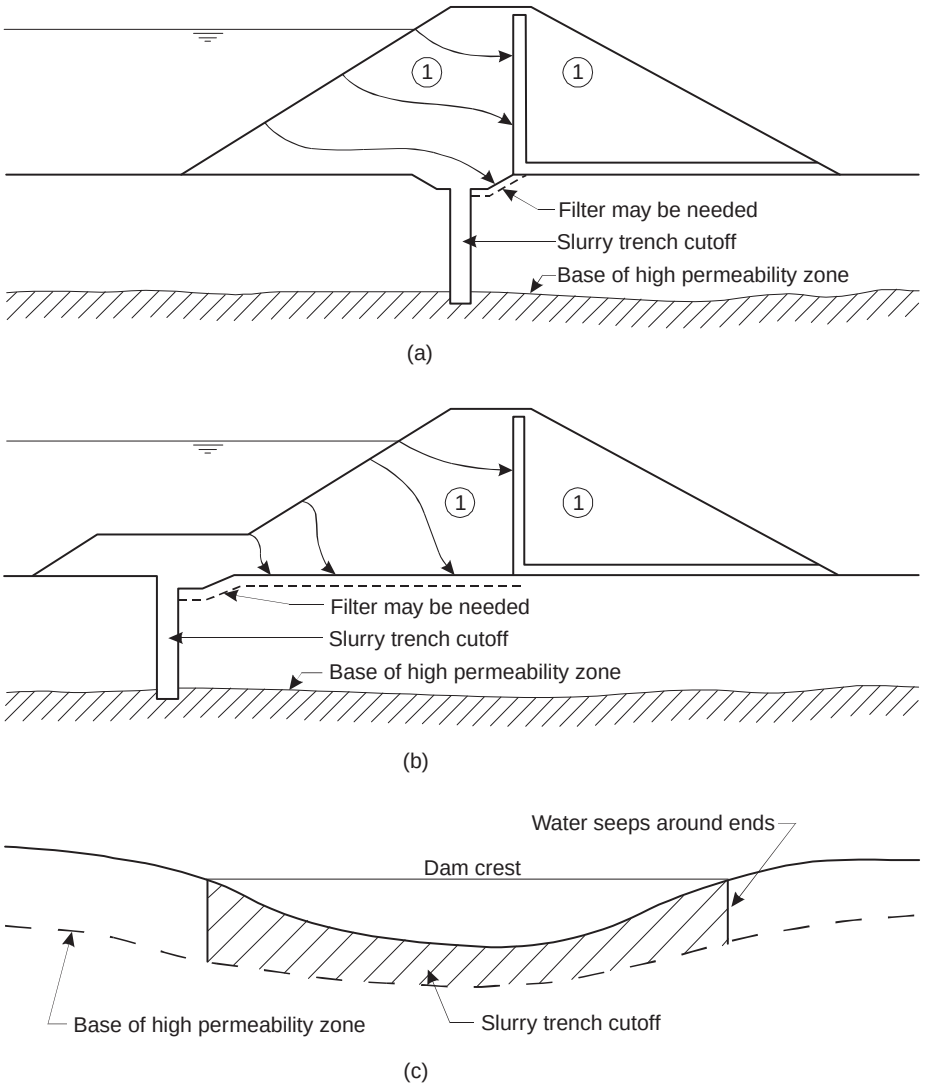


Figure 10.30 Slurry trenches and diaphragm walls cutoffs (a) Cutoff beneath the dam; (b) Cutoff upstream of the dam; (c) Potential seepage around the ends of the cutoff.

The practicality and economics of constructing the cutoff this way depend on:

- Whether dewatering is necessary to construct the cutoff.
- The availability of equipment to construct cutoffs by other methods, e.g. slurry trench.
- The stability of the sides of the trench during construction (which is dependent on soil type, shear strength and effectiveness of dewatering in controlling pore pressures).

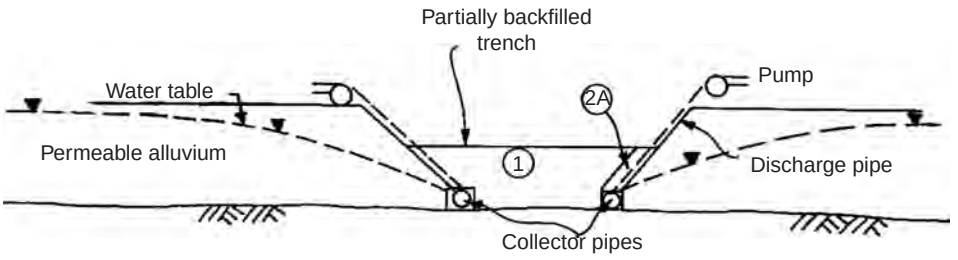


Figure 10.31 Dewatering system for cutoff trench.

In most cases the economic depth is likely to be less than 10 metres. Beyond this depth diaphragm walls and other wall cutoffs are likely to be more economic.

If dewatering of the permeable soil is necessary, it is likely that the dewatering has to continue while the trench is being backfilled with earthfill. Figure 10.31 shows a possible dewatering arrangement. However provision would have to be made for careful backfilling of the collector pipes with grout, and making the backfill around the pipes filter compatible with the Zone 1 and alluvium, or erosion may occur into the pipe or backfill.

Where a deep cutoff trench is required in the foundation, the geometry of the trench should be designed so that it is not narrow. This is to avoid the potential for hydraulic fracture though the cutoff trench due to arching effects. The side slopes of the cutoff trench should be no steeper than 45° to avoid this issue.

As shown in Figure 10.23, a filter zone may have to be incorporated into the downstream side of the cutoff trench to prevent erosion of the earthfill into the foundation.

If a cutoff trench can be constructed it does provide a very good quality cutoff, with a low permeability. For compacted clayey earthfill it would not be unreasonable to expect a permeability of 10^{-8} to 10^{-9} m/sec. The contact with the foundation can be of high quality and, for rock foundation, grouting can be carried out if necessary from the base of the trench.

10.8.3 Slurry trench cutoff backfilled with bentonite-sand-gravel

Slurry trench cutoffs consist of a continuous trench excavated by means of a backhoe, dragline or clamshell, or combination thereof, with the trench supported by bentonite slurry.

The trench is backfilled with sand or sand and gravel thoroughly mixed with the bentonite slurry. Often the sand and gravel will be from the excavation. Figures 10.32, 10.33 and 10.34 show the technique.

The slurry trench is constructed 1 m to 3 m wide. The maximum depth depends on the method of excavation. ICOLD (1985) and Xanthakos (1979) indicate that practical maximum depths are:

– Backhoe (excavator) 10 m,
– Dragline 25 m maximum, 20 m preferred maximum,
– Dragline and clamshell 25 m for slurry trench, cutoffs.

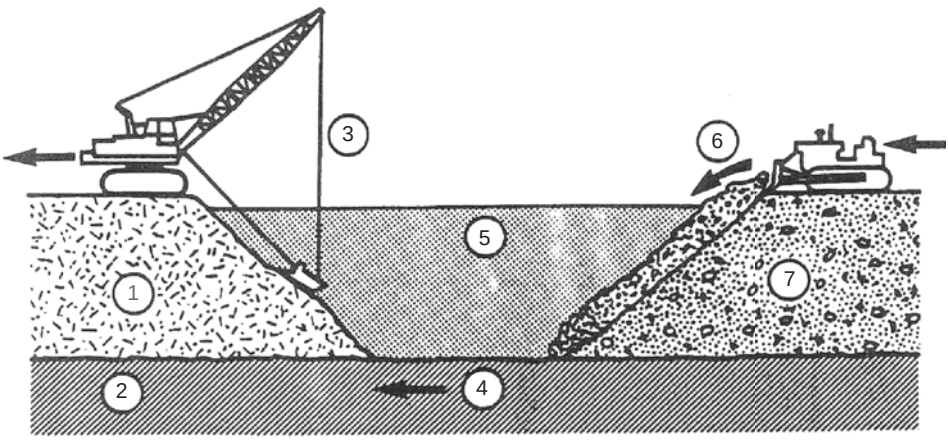


Figure 10.32 Slurry trench technique (1) Permeable virgin ground, (2) Substratum, (3) Excavation, (4) Direction of progress, (5) Trench filled with bentonite slurry, (6) Filling with aggregate, (7) Finished trench (ICOLD, 1985).

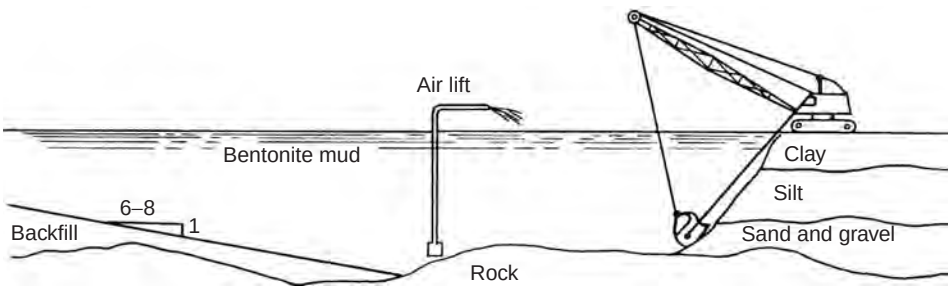


Figure 10.33 Slurry trench excavation by dragline (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

Note that it may be difficult to form a low permeability connection between the cutoff and the underlying low permeability stratum, particularly if the surface is irregular. An air lift or clamshell or scraper should be used as shown in Figures 10.33 and 10.34 to assist in cleanup of loosened debris at the base of the trench.

ICOLD (1985) indicates that the backfill will normally consist of bentonite slurry, with 5–15% bentonite (by weight), a Marsh funnel viscosity of greater than 40 seconds, mixed with well-graded sand and gravel between 0.02 mm and 30 mm size. The mixture should have a standard concrete cone slump of 100–200 mm. Xanthakos (1979) indicates that naturally occurring clays from the site may be used, although they are unlikely to be suitable where their liquid limit is greater than 60% and may be difficult to mix if they remain in lumps. Table 10.8 gives particle size distributions for the

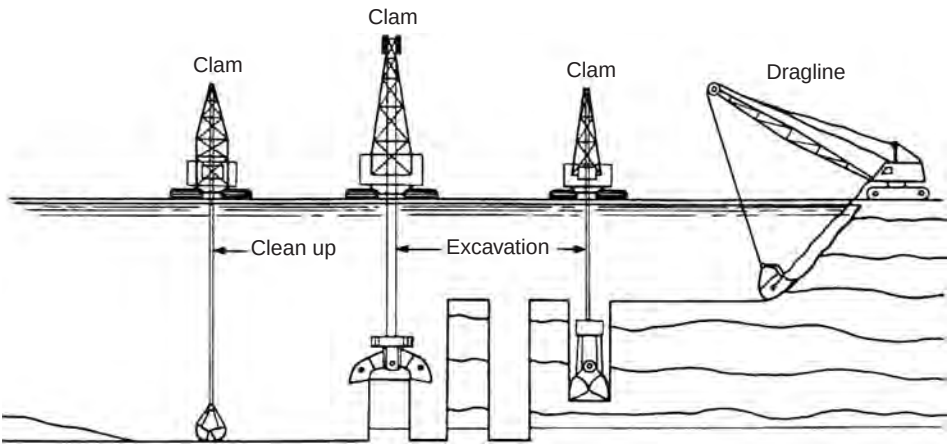


Figure 10.34 Slurry trench excavation by dragline and clamshell (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

Table 10.8 Typical gradation limits for slurry trench backfills (adapted from Xanthakos, 1979).

	Percentage passing	
Sieve size (mm)	*UK and Australian cutoffs*	*USA cutoffs*
75	80–100	80–100
19	40–100	40–100
4.75	30–70	30–80
0.6	20–50	20–60
0.075	10–25	10–30

combined backfill material quoted by Xanthakos. Xanthakos indicates that at least 10% silt and clay fines passing 0.05 mm is needed to give adequately low permeability.

The backfill may be mixed by withdrawing, dozing and blading the excavated soil to remove lumps of clay and silt or pockets of gravel. The soil is then mixed with the bentonite slurry, either by using the same earthmoving equipment or, definitely preferably, by using a concrete batch plant type mixer to ensure a uniform product.

The backfill is placed into the trench by gradually pushing it in by bulldozer, so that the backfill slope is between 6 H to 1 V and 8 H to 1 V. To start backfilling, the initial slope should be formed by lowering the backfill using (say) a clamshell bucket, as dropping the backfill through the slurry would cause segregation. A gap of 15 m to 45 m is left between the excavation face and toe of backfill to ensure proper cleanup of the trench base can be carried out.

No attempt is made to densify or compact the backfill.

Xanthakos (1979) indicates that in some cases 0.6–0.9 m of concrete may be placed at the bottom of the trench to provide protection against erosion of the base under seepage flows.

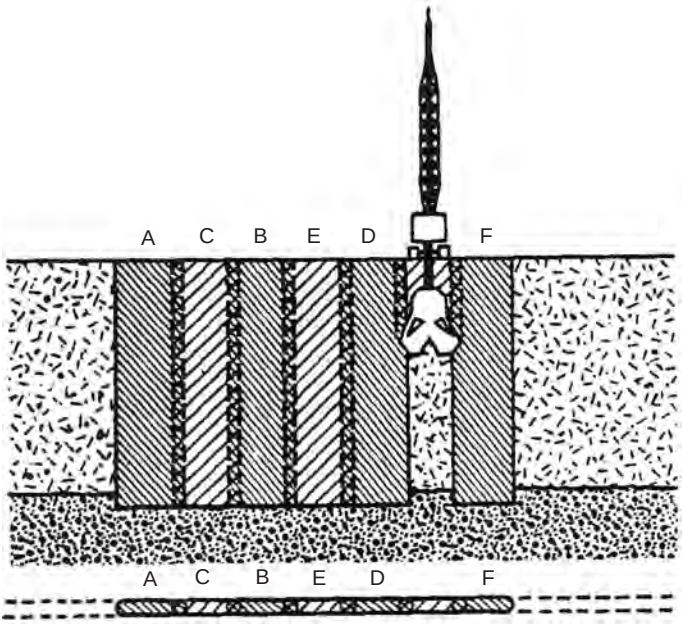


Figure 10.35 Grout diaphragm wall, (1) Order of construction of panels A, B, C, D, E, F, G; (2) ABDF primary panels, CEG secondary panels (ICOLD, 1985).

The top of the trench must be protected from drying by placing earthfill over the top. Settlement of the cutoff is quoted by Xanthakos (1979) as being between 25–150 mm for trenches 15 m to 25 m deep and 25 m wide. Less settlement is observed in narrower trenches. Settlement mostly takes place in the first 6 months.

The authors would not adopt this type of cutoff for a critical situation because of the difficulties in maintaining quality control of mixing and placement.

10.8.4 Grout diaphragm wall

Grout diaphragm walls are excavated continuously in panels as shown in Figure 10.35, with the trench supported by cement/bentonite slurry. This slurry is left in the trench and cures to give a low strength, low permeability compressible wall. The panels are excavated in the sequence shown in Figure 10.36, with the secondary panels being excavated before the slurry has hardened excessively in the primary panels but has hardened sufficiently to be self-supporting. This obviates the need for end support for the panels as is used for cast in place diaphragm walls (see Section 10.8.5).

The trench may be between 0.5 m and 1.5 m wide (ICOLD, 1985) but will often be nearer the narrow limit for economy provided adequate control of verticality of panels can be ensured to maintain overlap between panels.

Excavation is carried out by grab buckets or clamshells. While ICOLD (1985) make no definitive statement on maximum practical depth, it can be inferred that

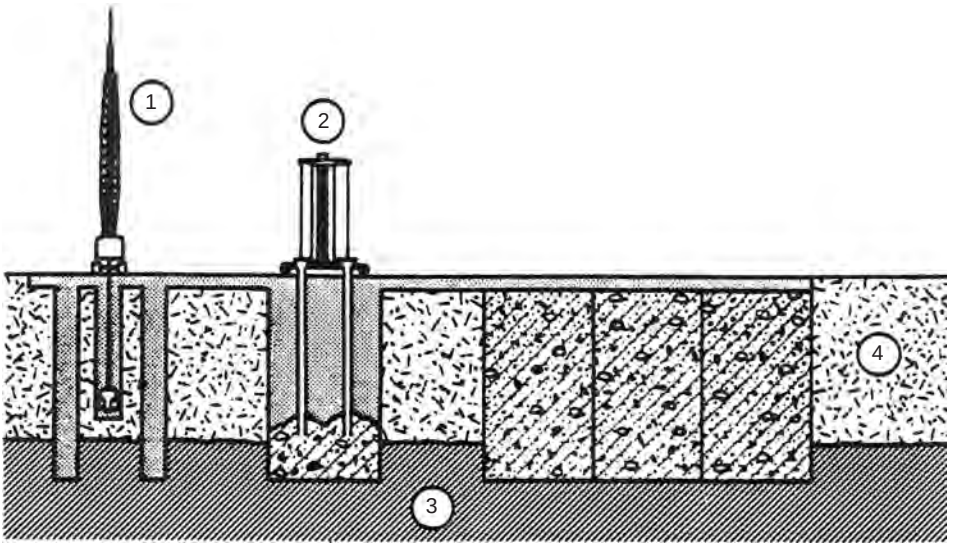


Figure 10.36 Diaphragm wall cutoff, (1) excavation; (2) concreting; (3) substratum; (4) permeable layer (ICOLD, 1985).

grout diaphragm walls may be used to at least 50 m depth. The authors have been involved in projects where 40m has been achieved.

(ICOLD, 1985) indicate that the cement/bentonite grout will usually have the following composition per cubic metre of grout:

- 80–350 kg cement,
- 30–50 kg bentonite.

Xanthakos (1979) indicates that typical mixes will be:

- 15–20% cement,
- 2–4% bentonite,
- 5–10% sand and gravel.

The water cement ratio (by weight) will be between:

- 4:1 and 10:1 for ground granulated blast furnace cement (BLF),
- 3.3:1 and 5:1 for Portland cement (P).

The blast furnace cement has greater resistance to attack by aggressive groundwater (pore water) which dissolves the free lime in the cement and selenitic water. This attack by aggressive water results in the formation of tricalciumsulpho-aluminates which destroys the hardened grout by expanding. An alternative to BLF would be fly ash, which may be added in proportion 10% to 100% by weight of cement (ICOLD, 1985).

Retarders are added to the mix to control the curing process mainly to delay the initial set.

The addition of cement, which has free lime ($Ca(OH)_2$), and/or gypsum ($CaSO_4$) to the bentonite slurry makes it flocculate because of cation exchange of the Ca^{++} ions for Na^{+} ions in the bentonite. The slurry remains stable (i.e. does not bleed) but does not form such an effective filter cake on the sides of the trench as bentonite and hence losses are greater. Xanthakos (1979) indicates losses may be up to 100% of the trench volume. This increases costs but gives a greater effective wall width. Admixtures can be used to reduce the flocculation effect and the use of BLF cement (with less free lime) instead of Portland cement assists.

The cement bentonite grouts commonly have very low strength compared to concrete. ICOLD (1985) indicate an unconfined compressive strength of 100 kPa at 28 days, 150 kPa at 90 days. The strength is affected by water/cement ratio and cement type. The grout is able to withstand considerable plastic deformation to accommodate settlement due to embankment construction. It is best if the cement-bentonite grout has a Young's Modulus just a little larger than the surrounding soil. In this way the load applied by the dam causes the wall to remain in compression.

A potential issue with grout diaphragm and soil-bentonite walls is that the cement/bentonite may be eroded if cracks form in the wall and high seepage gradients are present along the cracks. This is discussed further in Section 10.8.7.

10.8.5 Diaphragm wall using rigid or plastic concrete

Diaphragm walls are excavated in alternating panels as shown in Figure 10.36 with the panel supported by bentonite. The wall is constructed by tremie pipe placement of concrete or cement-bentonite concrete ("plastic concrete").

A steel "stop-end" tube as shown in Figure 10.37 supports the ends of each panel. The pipe is removed after initial set of the concrete leaving a half round key which is used as a guide for the excavating tool, thus reducing potential misalignment of panels and leakage.

It should be noted that for dam cutoffs the walls are not usually reinforced with steel.

The steel pipe for the stop-end should have a diameter equal to the trench width, so that concrete does not leak past the tube. Figure 10.38 shows the effect of overbreak in a gravel layer, allowing concrete to surround the tube and make it difficult to remove the tube.

Other joint systems may be used to achieve a better contact between adjoining panels and improve water tightness. Some examples are given in Figure 10.39. Millet *et al.* (1992) give other examples.

The trench wall thickness is generally 0.6 m for walls up to about 30 m deep, increasing to 1 m or 1.2 m for deep walls e.g. 50 m. The added width is required to assist in maintaining overlap between adjacent panels. The usual specified tolerance for verticality is 1/100 or 1/200, with some instances of 1:500 being required (Xanthakos, 1979), Tamaro and Poletto (1992), Millet *et al.* (1992).

Excavation is carried out by clamshell, scraping bucket or rotary drilling equipment. Details are given in Section 10.8.6.

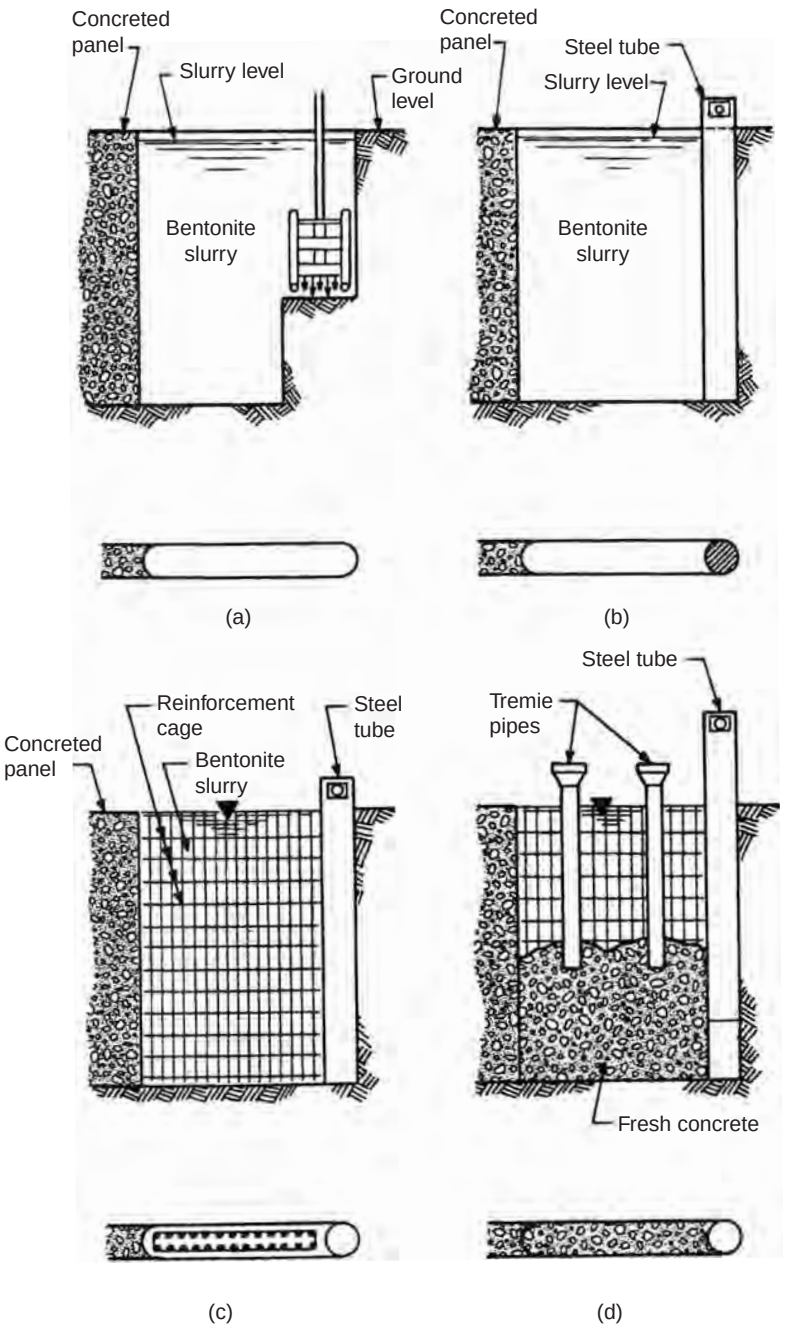


Figure 10.37 Typical construction sequence of a diaphragm wall executed in four stages: (a) Excavation; (b) Insertion of steel tubing; (c) Placement of reinforcement cage; (d) Concrete placement (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

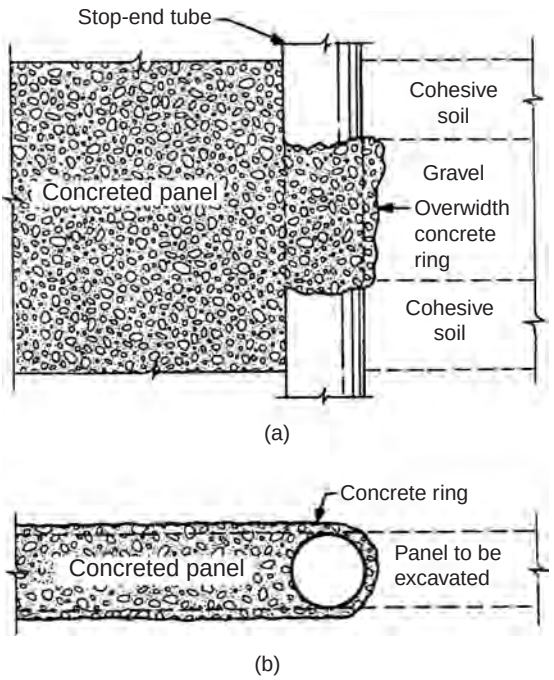


Figure 10.38 Penetration of concrete beyond the stop-end tube due to overbreak (a) Partial elevation; (b) Partial section (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

The walls may be constructed of conventional concrete and often are for building construction work. For dam applications, concrete is too rigid. As the soil mass surrounding the wall compresses under the weight of the embankment during its construction and the water load as the dam is filled, substantial loads will be shed onto the wall by negative skin friction. This can cause crushing of the wall and penetration of the wall into the dam fill.

For most dam applications it is preferable to use a plastic concrete backfill i.e. a concrete with bentonite added. This is sometimes called controlled low strength material (CLSM). ICOLD (1985) suggest that the following properties are desirable:

- The Young's Modulus of the wall should relate to that of the soil, i.e. $E_{wall} \leq 5E_{soil}$.
- Failure strain should be high, typically at least 5%, often 10% to 15%.
- Unconfined strength. A high strength is not important, in the order of 1–2 MPa (Xanthakos suggests an upper limit of 2 MPa).

ICOLD (1985) indicate that the composition of plastic concretes should be (per cubic metre):

- 400–500 litres – bentonite slurry,
- 100–200 kg – cement,
- 1300–1500 kg – well graded aggregates less than 30 mm size.

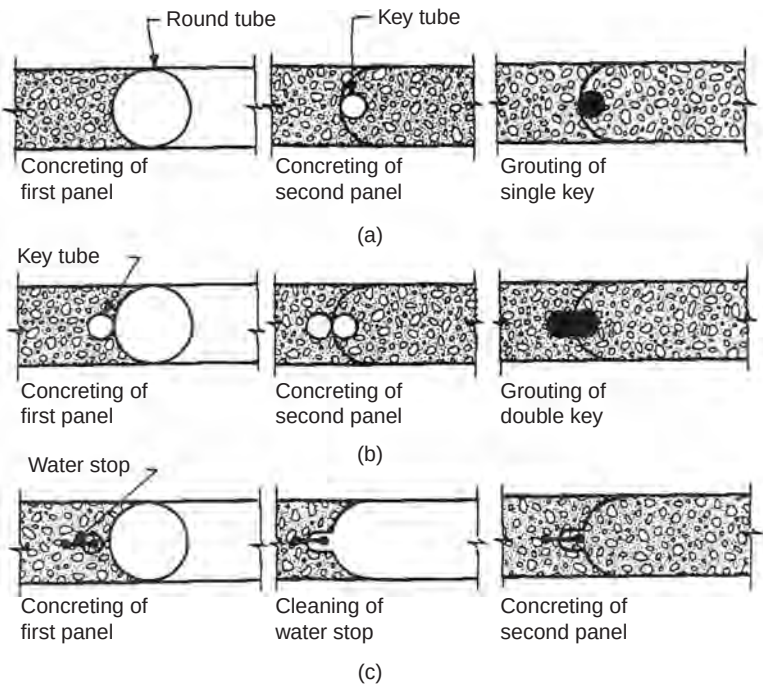


Figure 10.39 (a) Single key joint; (b) Double key joint; (c) Water stop joint (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

The water cement ratio will be between 3.3 to 1 and 10 to 1 according to the type of cement, the higher values being for BLF cement, the lower for Portland cement.

The bentonite slurry is to keep the cement and aggregates in suspension during placement and assure plasticity and low permeability. The percentage (by weight) of bentonite to water varies from 2% to 12% according to its hydration. The Marsh funnel viscosity should be 50 seconds.

If coarse aggregates are replaced by medium to fine sand the composition should be:

- 375–750 litres – bentonite slurry,
- 75–290 kg – cement,
- 500–100 kg – medium to fine sand.

Millet *et al.* (1992) emphasise the need to hydrate the bentonite before mixing in cement and aggregates.

The concrete (either conventional or plastic) must be placed in the wall by a tremie pipe to avoid segregation. Xanthakos (1979) indicates that it is desirable to complete the concrete pours in less than 4 hours to avoid any significant stiffening of the concrete. For panels up to 3.5–4.5 m long, a single tremie pipe is adequate but, for longer panels, two pipes may be needed.

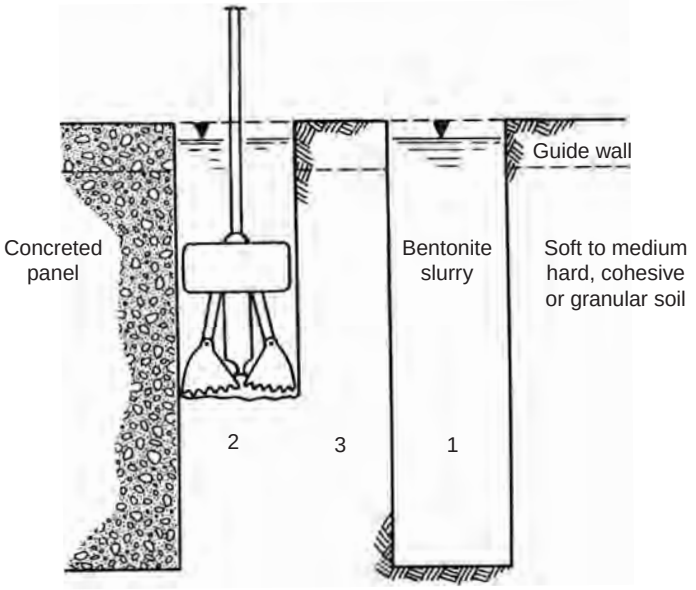


Figure 10.40 Single stage panel excavation with clamshell bucket: Passes 1 and 2 spread of clamshell bucket; Pass 3 spread of clamshell bucket minus clearance for grab to embrace soil (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

Reinforced concrete guide walls are constructed ahead of the trenching operation, for diaphragm walls filled with grout or concrete. The guide walls:

- Control the line and grade in the trench,
- Support the sides of the trench from heavy construction loads,
- Protect the sides of the trench from turbulence and erosion,
- Can be braced to support the top of the trench,
- Act as a guide trench for the slurry.

Xanthakos (1979) gives details of guide wall design.

10.8.6 Methods of excavation of diaphragm walls

Diaphragm walls are usually excavated as a series of panels, each panel as a series of passes as shown in Figure 10.40.

This method can be applied to soft to medium hard, granular and cohesive soils, provided there are not boulders or other obstructions. The minimum "pass" is about 2 metres, the maximum about 6 m.

Excavation will usually be carried out using clamshell type bucket as shown in Figure 10.40. The clamshells are of two types:

(a) Cable suspended which:

- Are easy to keep vertical,
- Are maneuverable,

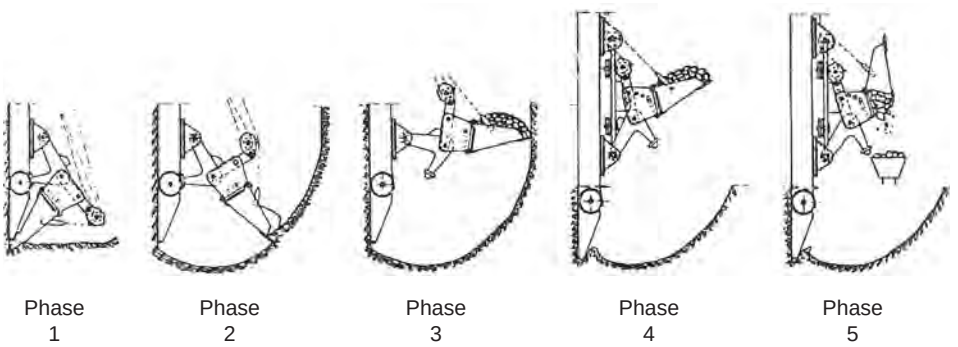


Figure 10.41 Action of a scraping bucket excavator (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

- Can work in limited headroom,
- Depend on their self weight to close the grab. Pulleys or hydraulic systems are used to assist in closing.
- Can be used up to 75 m (cable operated), or 55 m hydraulic.

(b) Kelly bar which:
- Are better suited to homogeneous soil,
- Have a maximum depth of 40 m (single piece Kelly) telescoping up to 60 m,
- Requires large headroom,
- Are hydraulically or power operated, with the Kelly bar weight assisting to close the bucket.

Other types of excavation equipment which can be used include:

(c) Bucket scraper. This is attached to a Kelly bar as shown in Figure 10.41.

(d) Rotary drills

In this technique the excavation is carried out by a drill which has several bits mounted on a submersible frame. Figure 10.42 shows one type of such equipment.

The rotary drill bits can excavate into soil or soft rock. The cuttings are suspended in the slurry and flushed from the trench, usually by reverse circulation, to improve the lifting capacity. Screens and settling tanks are used to remove the cuttings.

(e) Percussion tools. Percussion tools and rock chisels are used where boulders, cobbles or other hard materials are encountered. The broken pieces are removed by clamshell bucket. Figure 10.43 shows examples of this equipment.

In some applications where boulders are present it may be economic to use a two stage operation (Figure 10.44), where the first stage consists of excavating pilot holes with percussion rotary drill and the second stage removing the material between the holes with a clamshell bucket. The first stage holes are spaced to leave an intermediate piece 0.3 m to 0.6 m narrower than the clamshell bucket grab.

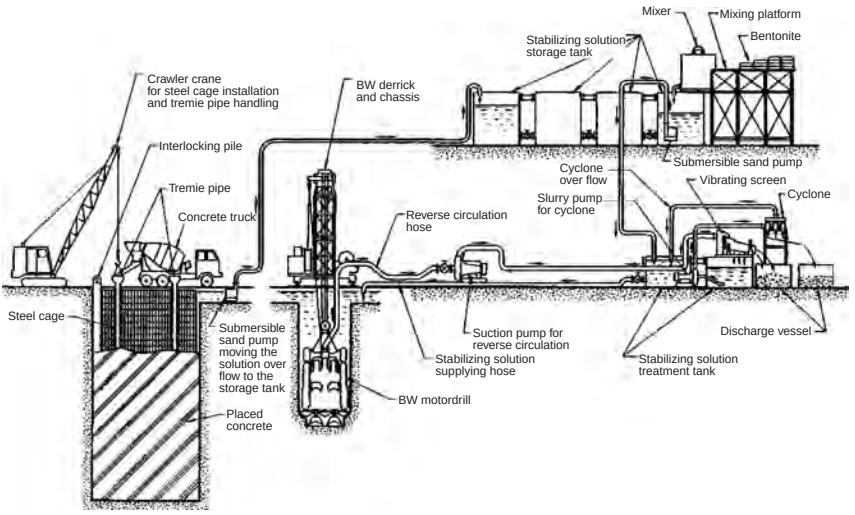


Figure 10.42 Rotary drill technique (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

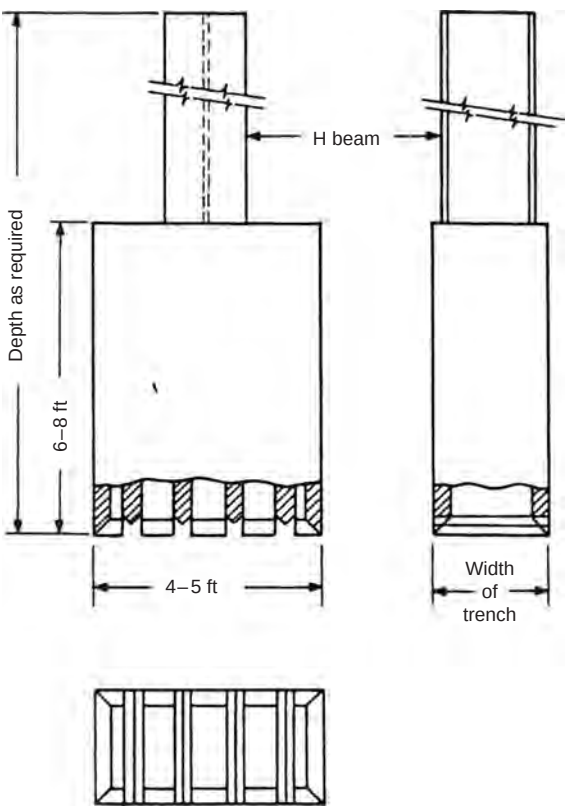


Figure 10.43 Typical chisel details for breaking embedded boulders in site excavations (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

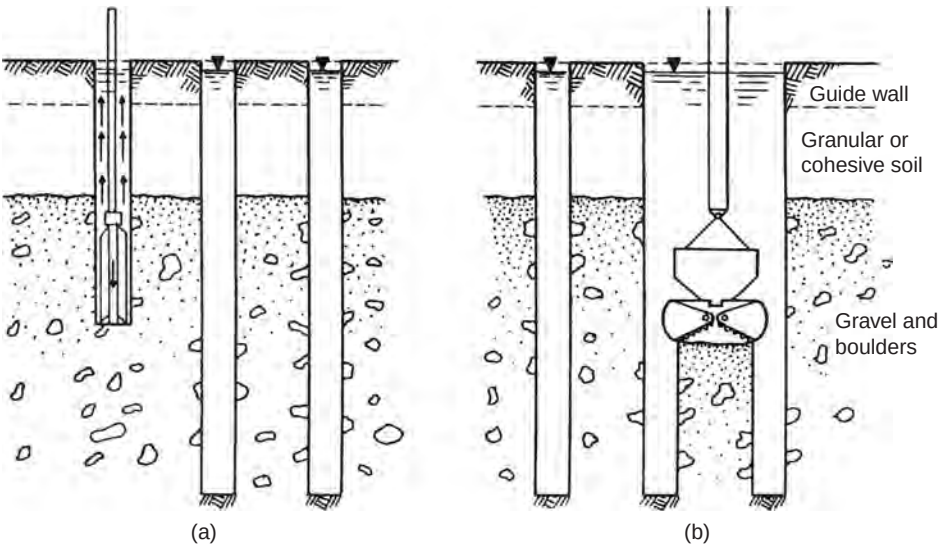


Figure 10.44 Two stage panel excavation with percussive tools and clamshell grab: (a) Excavation of pilot holes with percussive tools; (b) Panel excavation with clamshell bucket (Xanthakos, 1979, reproduced with permission of McGraw-Hill).

(f) Hydrofraise – which is a down-the-hole reverse circulation rig which excavates a 2.8 m long panel. The use of this technique for Mud Mountain dam is described in Davidson *et al.* (1992). They also describe the use of grouting to control hydraulic fracture in the dam core as the slurry wall was built.

Where bentonite mud is used to support an excavation, as required for virtually all the methods described above, it should be noted that the chemical composition of the ground-water will have an effect on the performance of the bentonite slurry.

In particular, groundwater with a high salt content (i.e. NaCl) or a high calcium ion concentration will cause the bentonite to flocculate. This is caused by contraction of the diffuse double layer around the clay particles as described in Chapter 7.

ICOLD (1985) indicate that:

– Fresh water should be used to make the bentonite slurry. Water with up to 5 g/litre of salt may be used, but the bentonite content will have to be increased;
– "Colloid protectors" (called peptizers or dispersing agents by Xanthakos, 1979) may be used to prevent flocculation.

Xanthakos (1979) indicates that even if trenching into soil with groundwater of high salts content, even of seawater quality, the slurry only takes up ionic salt and the flocculation can be controlled. If only saline water is available, attapulgite may be substituted for bentonite.

More information on muds can be obtained from Xanthakos and from Chilingarian and Vorabutr (1983).

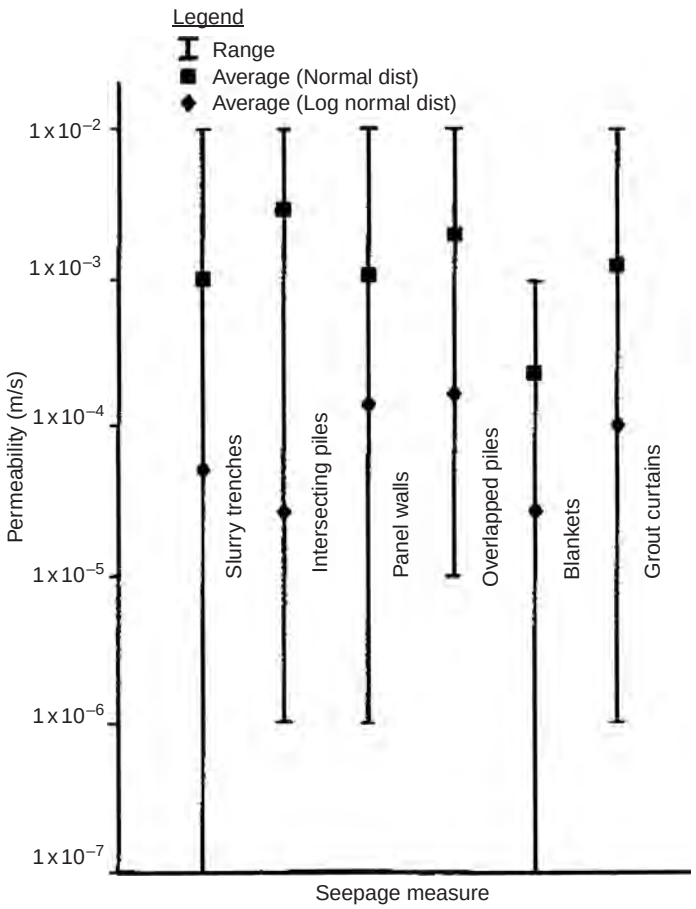


Figure 10.45 Permeability of seepage control measures (Powell and Morgenstern, 1985, reproduced with permission of ASCE).

10.8.7 Permeability and performance of cutoff walls

ICOLD (1985) indicate that the permeability's which can be achieved for cutoff walls are:

- 10^{-7} to 10^{-8} m/sec – for grout walls,
- 10^{-8} to 10^{-9} m/sec – for plastic concrete walls,
- 10^{-9} to 10^{-10} m/sec – for concrete,
- compared to 10^{-6} m/sec for a grout curtain.

Millet *et al.* (1992) give similar values.

Powell and Morgenstern (1985) reviewed published case histories and concluded that the range of permeability's achieved was as shown in Figure 10.45. These are several orders of magnitude higher permeability than the ICOLD values.

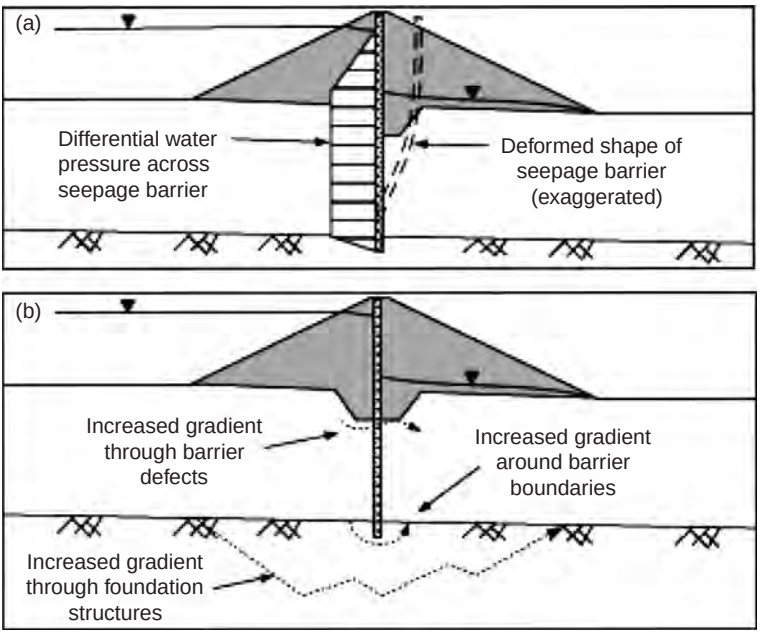

(a)
Differential water pressure across seepage barrier
Deformed shape of seepage barrier (exaggerated)
(b)
Increased gradient through barrier defects
Increased gradient around barrier boundaries
Increased gradient through foundation structures

Figure 10.46 Mechanisms in which increased water pressure and gradient affect seepage barrier or cutoff performance: (a) Deformation due to differential water pressure; (b) Increased gradients through and beneath the cutoff (Rice and Duncan, 2010a, with permission of ASCE).

Rice and Duncan (2010a, b) gathered data for seepage barriers (cutoffs) for 50 dams and analysed data for 30 of these. They found that the long term performance of the seepage barriers in dam foundations or constructed through dam embankments was affected by a number of mechanisms all relating to the fact that when a low permeability barrier is constructed in a more permeable strata high hydraulic pressures build up on the upstream side of the barrier and as a result there are increased and potentially high hydraulic gradients through and below the barrier where it is founded in the rock or low permeability strata. These mechanisms are shown in Figure 10.46.

They found for two concrete cutoffs in alluvium and one plastic concrete cutoff in the embankment the barrier had actual permeability in the range 10^{-4} to 10^{-6} m/sec whereas the expected permeability had been 10^{-7} to 10^{-8} m/sec. They attributed this to cracking forming in theses relatively rigid cutoffs due to the deflection of the walls induced by differential water pressures. They demonstrated that even small cracks could result in significant flows. If the wall is primarily intended as an internal erosion control measure, then provided the cracks are not too wide they are unlikely to be a cause of concern as the flows though the crack will be flow limited and hence stop the pipe enlarging.

They also warn that the high seepage gradients through and beneath the cutoffs can lead to erosion of soil infill in joints in rock into which the cutoffs are founded, and suffusion in internally unstable soils.

These are shown in Figure 10.46 and 10.47.

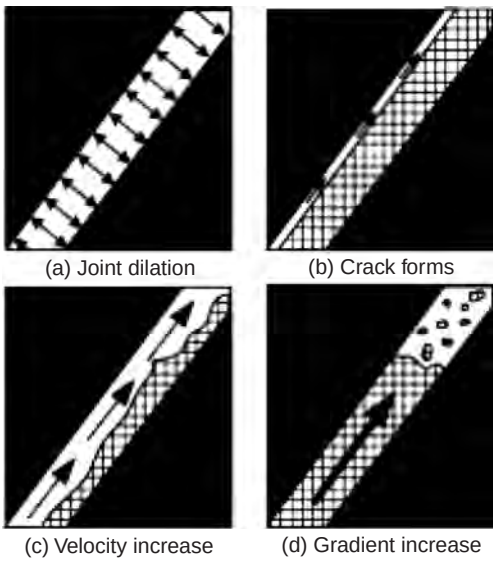


Figure 10.47 Mechanism of joint infill erosion associated with increased hydraulic pressure and gradient: (a) Joint dilation: (b) Crack deformation due to dilation; (c) Increased hydraulic velocity in open joint; (d) Increased gradient over closed joint (Rice and Duncan, 2010a, Reproduced with permission of ASCE).

The ICOLD (1985) permeability figures are more likely to reflect the laboratory properties of the materials in the cutoff wall, while the Powell and Morgenstern (1985) and Rice and Duncan (2010a, b) values reflect the equivalent permeability of the constructed wall including defects (defects caused by construction or deflection of the wall). The permeability of well-constructed walls may approach that given by ICOLD (1985) but designers should take care with detailing and constructors with quality control. They should not be too optimistic about what will be achieved.

The suggested permeability of 10^{-6} m/sec for a grout curtain by ICOLD (1985) is equivalent to 10 lugeons and is in the correct range for most jointed rock masses. This is discussed more in Chapter 18.

10.8.8 We live in a three dimensional world

We live in a three, not two, dimensional world. Figure 10.48 shows the foundation for a 20 m high dam for the Drayton Mine in the Hunter Valley for which the first author was responsible for the investigation and design. The grouting as shown was highly successful, closing to 1–3 lugeons. What was overlooked was that the dolerite sill day-lighted in the storage, was highly permeable due to jointing and water simply went around the ends of the grout curtain. The water level in the storage dropped by ≈25 mm/day. The water could be heard flowing in the dolerite sill in a borehole about 100 metres downstream!

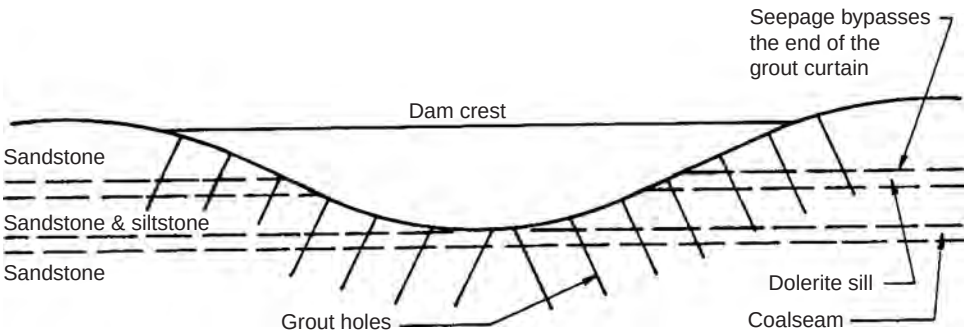


Figure 10.48 An example of seepage out-flanking a grout curtain.

Similar problems can occur with cutoffs into foundations of alluvial or lateritic soils (Figure 10.18) and it is often necessary to take the cutoff beyond where the crest of the dam meets the excavated foundation surface for the embankment.

Another example of a 3 dimensional problem is seepage from tailings dams is given in Figure 19.33 which shows the plan of the tailings storage for the Boddington Mine, Western Australia. At the time of the first author's initial involvement, the emphasis on estimating the seepage from the storage had been on the seepage under the dam structures. The foundation rock was deeply weathered and lateritised and likely to have a high mass permeability. Hence seepage would occur through the hills to the north and east, not just under the dams. The groundwater table to the south was towards the dam. Being a tailings dam, the tailings themselves can control the seepage, except where the water on the tailings is in contact with the foundation.

10.9 EXAMPLES OF DAM UPGRADES TO ADDRESS DEFICIENCIES IN INTERNAL EROSION AND PIPING CONTROL

10.9.1 Upgrades to reduce the likelihood of continuation of erosion by providing filters and cutoffs

Figures 10.49 to 10.52 show examples of dam upgrades which were carried out to address deficiencies in the control of internal erosion and piping by upgrading the filters.

In some cases the upgrades also address flood and potential liquefaction in the dam foundation. One or more of the authors has been involved in these projects.

The following are some details of these upgrades:

Wingecarribee Dam (Figure 10.49)

This upgrade for this 1970s' dam consisted of the excavation of a 3 m deep × 1.2 m wide trench to intercept the sloping chimney filter which had been designed and constructed only to reservoir full supply level. This was common practice up to and in a few cases beyond that time. Many dam engineers over this time did not appreciate

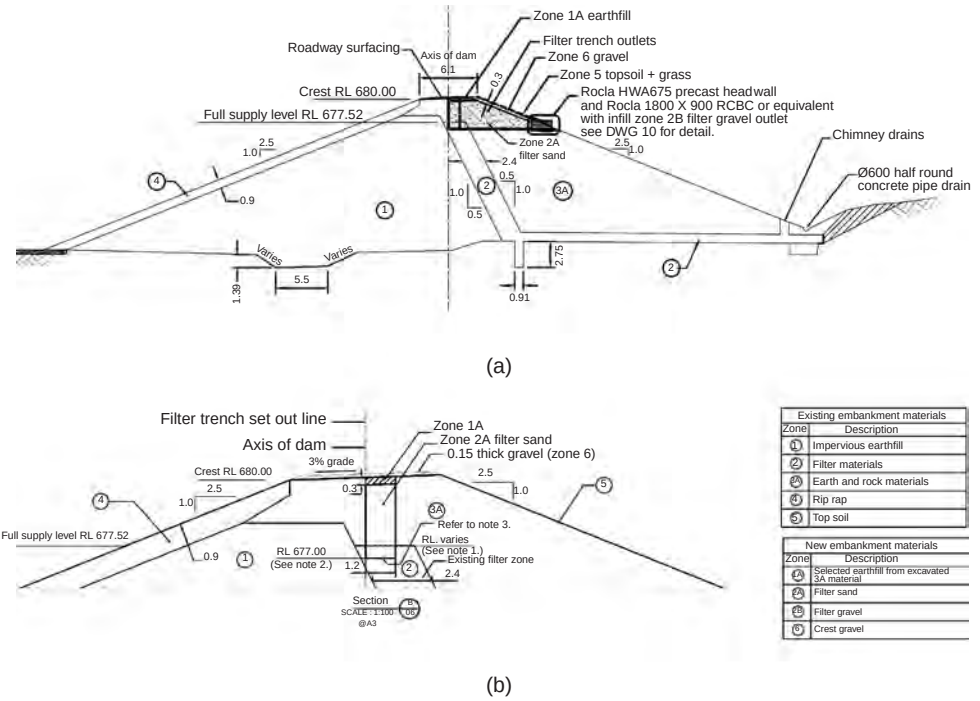


Figure 10.49 Extension of chimney filter, Wingecarribee Dam. (a) Typical section. (b) Details of filter extension. Courtesy of Sydney Catchment Authority and URS Australia.

the fact that internal erosion in plastic soils such as used for this core would erode in cracks formed by differential settlement and/or desiccation and this could develop in hours or days if the reservoir rose under flood conditions.

The new chimney filter extension was provided with transverse outlet trenches at 50 metre intervals. These are adequate because if internal erosion occurs the erosion will cause the eroded soil to form a "cake" on the filter and flow of water in the crack will be almost fully stopped.

The filter was compacted in the trench by vibrating plates supported on an excavator.

Toorourrong Dam (Figure 10.50)

Toorourrong dam is a puddle core earthfill dam constructed in 1885. It experienced several piping incidents in the 1970s. It is founded on a very complex alluvial deposit, some of which would be susceptible to liquefaction under large earthquakes. The dam also had insufficient spillway capacity to pass the design floods. Details are given in Johnston and Arnold (2011).

The upgrade consists of raising the dam with a concrete wave wall and embankment raise; ground improvement of the liquefiable foundations by construction of grouted stone columns; and provision of filters and drains to control internal erosion in the embankment and the foundation.

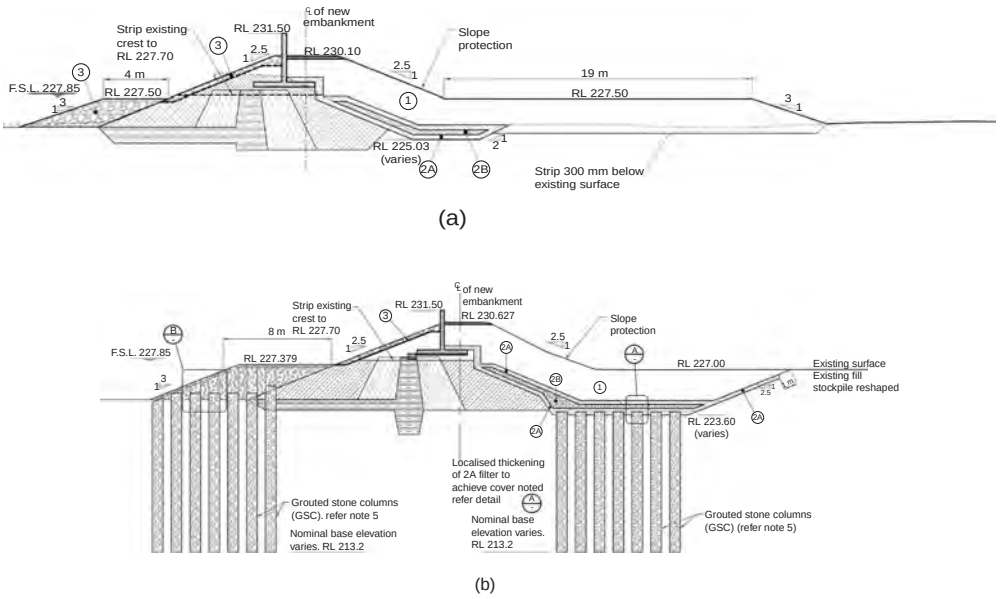


Figure 10.50 Upgrade of Toorourrong Dam for flood, seismic loads and internal erosion and piping. (a) Typical section. (b) Section where there were more extensive potentially liquefiable soils (Johnson and Arnold, 2011, Courtesy of Melbourne Water and the Water Resources Alliance).

The stone columns can potentially act as drains and because the foundation soils were very fine they would have required a fine filter backfill to prevent internal erosion of the foundation soil into the columns. This was too fine to be constructed using the equipment available so grouted stone columns were used. These provided the additional shear strength required in the foundation but did not allow erosion into the columns.

The very wide berms or each section were required to achieve the required factor of safety for the post earthquake liquefied soil condition.

Laanecoorie Dam (Figure 10.51)

Laanecoorie Dam is a puddle core earthfill dam constructed from 1889 to 1891. It breached by overtopping during a flood in 1909 and reconstructed.

During major floods in 2010 and 2011 the embankment cracked and deformed largely because the downstream slopes were partly inundated by high tailwater levels causing collapse settlement of the shoulder fill.

Investigations also found evidence of cracking and root holes through the core and the dam was upgraded in 2013 to address potential instability and internal erosion and piping.

The dam was buttressed with a rockfill berm with a two stage filter between the existing dam fill and the rockfill. Filters were also provided to control internal erosion of the foundation into the rockfill.

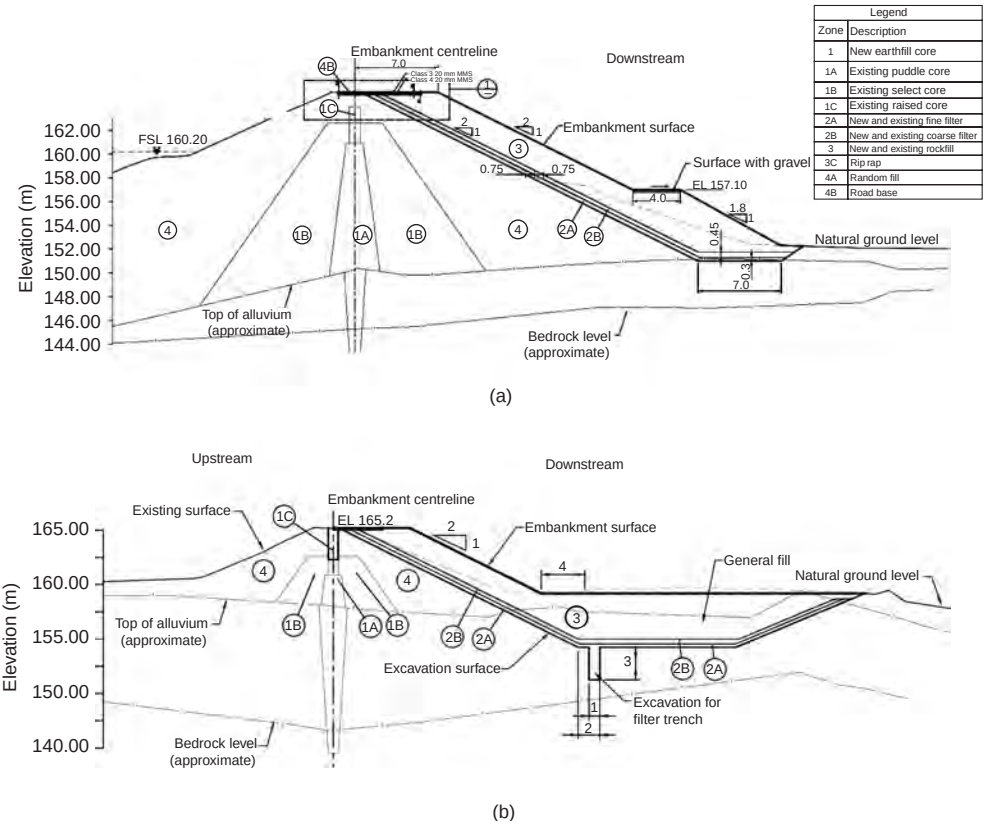


Figure 10.51 Upgrade of Laanecoorie Dam for internal erosion and piping. (a) Typical section in river bed. (b) Typical section in abutment (courtesy of Goulburn-Murray Water and URS Australia).

In the dam abutments a vertical trench up to 3 metres deep was excavated and backfilled with fine filter. This was provided to further control internal erosion in the foundation particularly in the upper 2 m to 3 m where roots from trees had been found in test pits.

It will be noted that the filters were constructed at a minimum practicable width of 0.75 m because of the high cost of filter materials. The filters were placed by excavator bucket progressively as the fill was raised. On other projects "Flo-Con" equipment has been used. This narrow width can only be achieved with very strict construction supervision.

On some projects segregation of the coarse filter has been a problem requiring very careful placement and to accommodate the coarser grading a minimum width of 1.0 metre or more has been adopted.

The use of rockfill rather than earthfill has been found to be the best solution in a number of dam upgrades because it is easier to place the rockfill in the narrow widths required, and placement is less affected by wet weather.

William Hovell Dam (Figure 10.52)
William Hovell dam is a 35 m high central core earth and rockfill dam constructed in 1971. The dam was designed and constructed to modern dam design standards but as for Wingecarribee Dam the Zone 2A fine filter was only taken to the full supply level, so the upper part of the dam had Zone 2B coarse filter, which did not meet no-erosion filter requirements for the impervious eathfill zone.

The dam also had inadequate flood capacity so was upgraded in 2010 to increase the flood capacity by raising the crest with a concrete parapet wall and upgrading the filters to provide Zone 2A and Zone 2B filters for the full height of the dam.

Parapet walls like this are often now constructed as precast units. They are embedded in the fill to provide adequate stability under floods up to the top of the parapet wall. The fill over the horizontal leg of the wall is sufficiently deep so heave will not occur even if full reservoir pressure develops below the full width of the wall base. Such pressures can develop if there is a slight rotation of the wall when water load is applied to the wall. The Zone 2C acts as a filter to control potential internal erosion through the Zone 1A fill at the parapet wall interface.

As for the Laanecoorie upgrade the filters were quite narrow and the same comments apply. The "traffic control bund" was reduced in size and Armco railing added during construction.

Buffalo and Eppalock Dams (Figures 10.53 and 10.54)
Upgrades of Buffalo and Eppalock Dams are described in Newman and Foster (2006) and Davidson *et al.* (2000). These are central core earth and rockfill dams constructed in the 1960s. They are characterized by relatively steep side slopes and the use of dumped rockfill. These resulted in significant settlement and longitudinal cracking of both dams. For Buffalo there was a step in the abutment over which a crack formed probably due to cross valley differential settlement during construction.

Figures 10.53 and 10.54 show the dam cross section and a schematic diagram showing the cracking.

Both dams were upgraded for internal erosion and piping and marginal stability of the upper slopes. For both the existing fine filter zone contained an excess of cohesive fines and so the fine filter could hold a crack. The coarse filter zones also did not satisfy no-erosion filter criteria but did satisfy excessive erosion criteria so the filters would be expected to seal the erosion with only small leaks which could be accommodated by the downstream rockfill zone.

It was demonstrated by quantitative risk assessment the upgrade of only the upper part of the dam was required to give a tolerable risk situation.

Hinze Dam Stage 3 (Figures 10.55 and 10.56)
The Stage 3 upgrade of Hinze Dam is described in Hunter and Toose (2009), Hunter *et al.* (2009), Obrien *et al.* (2009) and Davidson and Dann (2011). This involved construction of a 230 m long, 53 m deep and 0.83 m wide cast under bentonite low strength plastic concrete cutoff wall through the right abutment which was deeply weathered, highly permeable and had open defects due to laterisation. The dam was raised by 15 m as had been planned when the dam was first constructed. Figure 10.55 shows the maximum section. Deep excavation into Stage 2 was required at the crest to connect to the Stage 2 core and filters. Complex foundation treatment was also required to control internal erosion and piping in the foundation beyond the cutoff wall. This involved removing part of Stage 2 fill as shown in Figure 10.55.

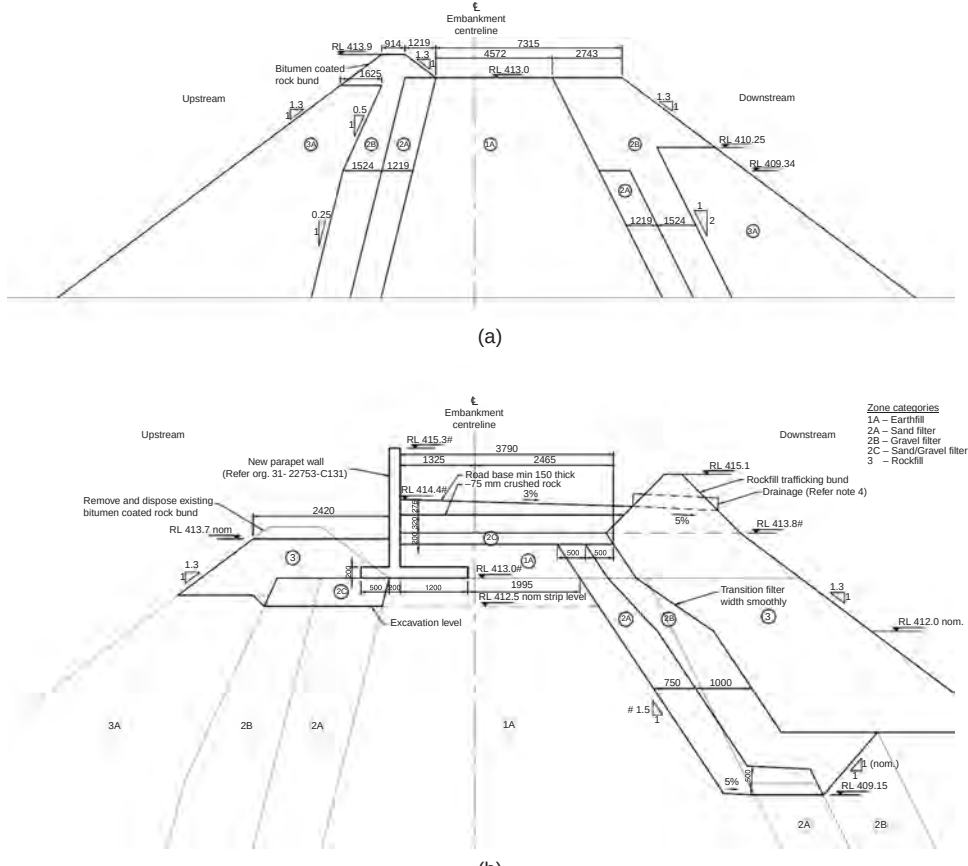


Figure 10.52 Upgrade of William Hovell Dam for flood and internal erosion and piping. (a) Typical; section prior to upgrade. (b) Typical upgrade section (courtesy of Goulburn-Murray Water and GHD).

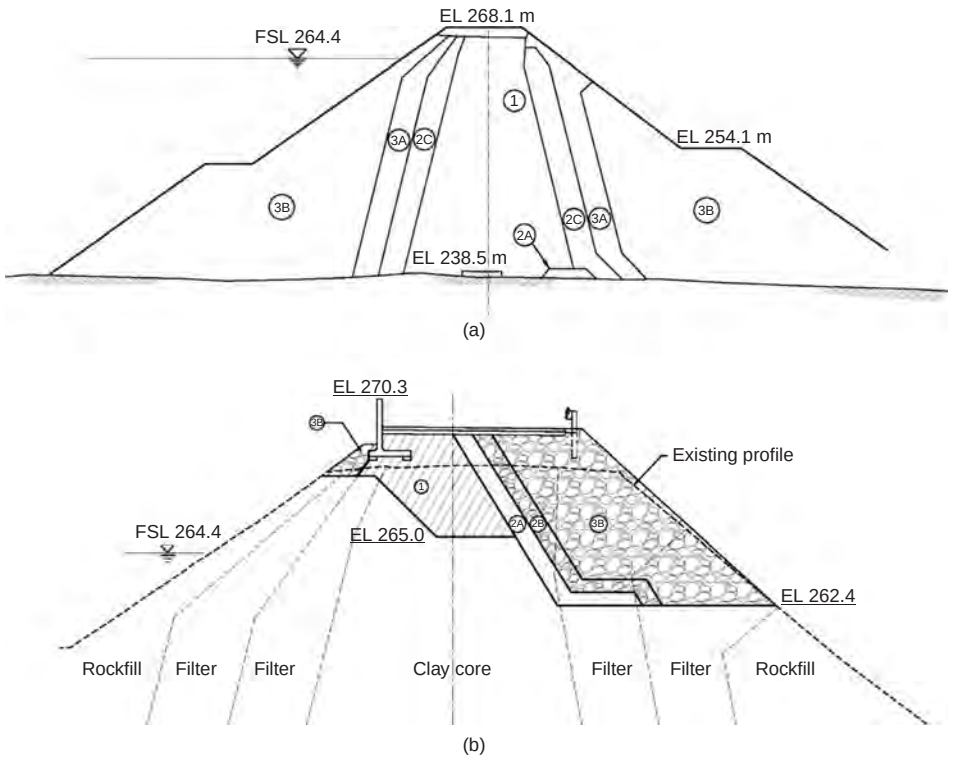


Figure 10.53 Upgrade of Buffalo Dam. (a) Dam as constructed. (b) Upgrade of crest of the dam (Newman and Foster, 2006).

10.9.2 Upgrades to reduce the likelihood of breach

As discussed in Section 8.11.4 the likelihood of breach by unravelling of the downstream rockfill for dams, which have large leakage flows before the filters seal, can be reduced by providing a berm constructed of large diameter rockfill.

This has been used in a number of dams particularly in Sweden and Norway over the last 10 to 15 years. These dams are usually central core earth and rockfill dams. The cores are glacial till and the downstream filter was often constructed of tunnel spoil or spoil from underground power stations.

As discussed in Nilsson (2007a, b, 2009) and Ronnqvist (2010), the filters were often too coarse to satisfy no-erosion criteria and fell into the some or excessive erosion range. Many of the dams had experienced internal erosion and piping incidents with significant leakage flows before the filters sealed.

To address this problem, the concept of the "Swedish berm", a rockfill berm on the dam's downstream rockfill shoulder, was introduced. This rockfill berm was designed to control the potentially large flow through the dam that might result from substantial erosion of the core. The flows through the dam are estimated assuming the core is eroded such that the permeability of the damaged core is the same as the

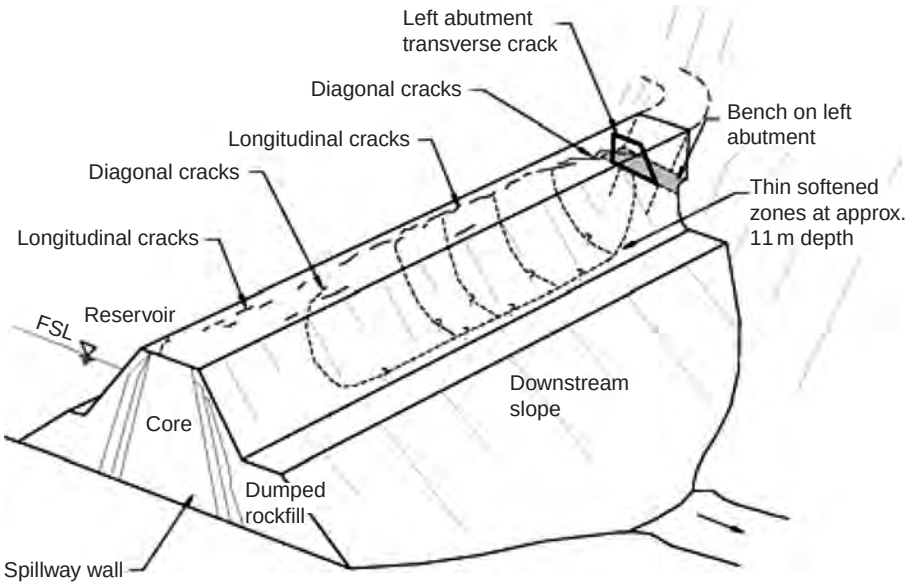


Figure 10.54 Schematic diagram of Buffalo Dam showing longitudinal and transverse cracking of the dam core (Newman and Foster, 2006).

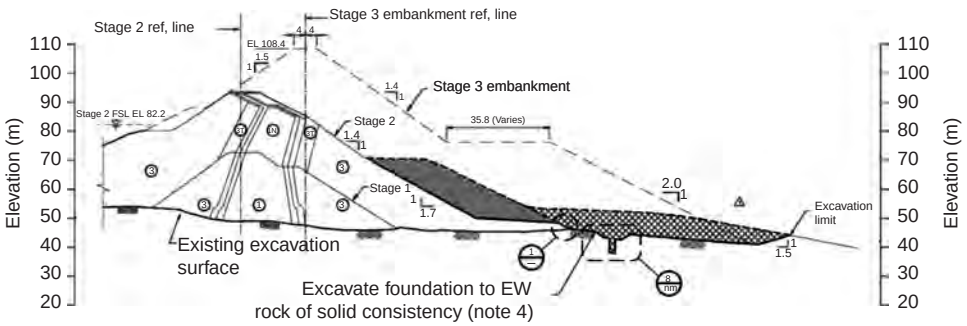


Figure 10.55 Hinze Dam Stage 3 right abutment section. Hatched and grey areas had to be excavated from Stage 2 to allow filters to be constructed on the foundation (Hunter and Toose, 2009).

rockfill and the flow through the dam is controlled by the upstream and downstream rockfill zones.

Figure 10.57 shows the assumed seepage conditions. The permeability of the downstream and upstream rockfill is determined from the gradation of the rockfill. From these the flow rate for which the berm is designed can be calculated. Details of the design method are given in EBL (2005).

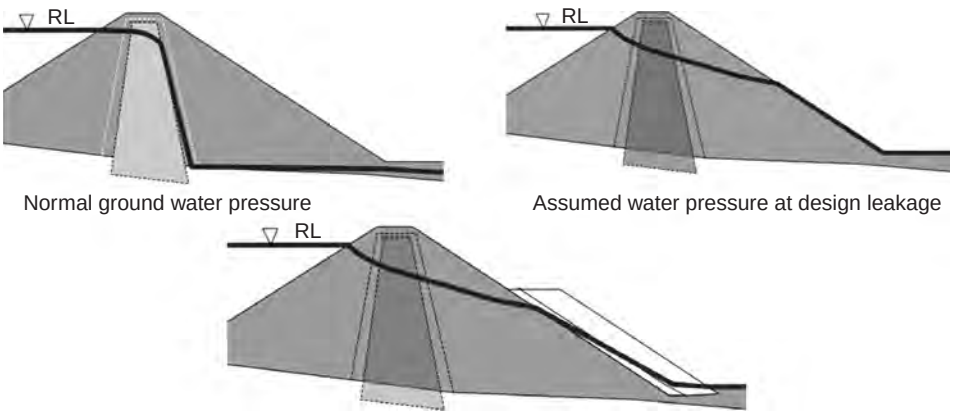

RL
Normal ground water pressure
RL
Assumed water pressure at design leakage
RL
Assumed water pressure at design leakage and strengthened by a toe berm

Figure 10.56 Assumed phreatic surfaces within the embankment for design of "Swedish berm" (Nilsson, 2009).

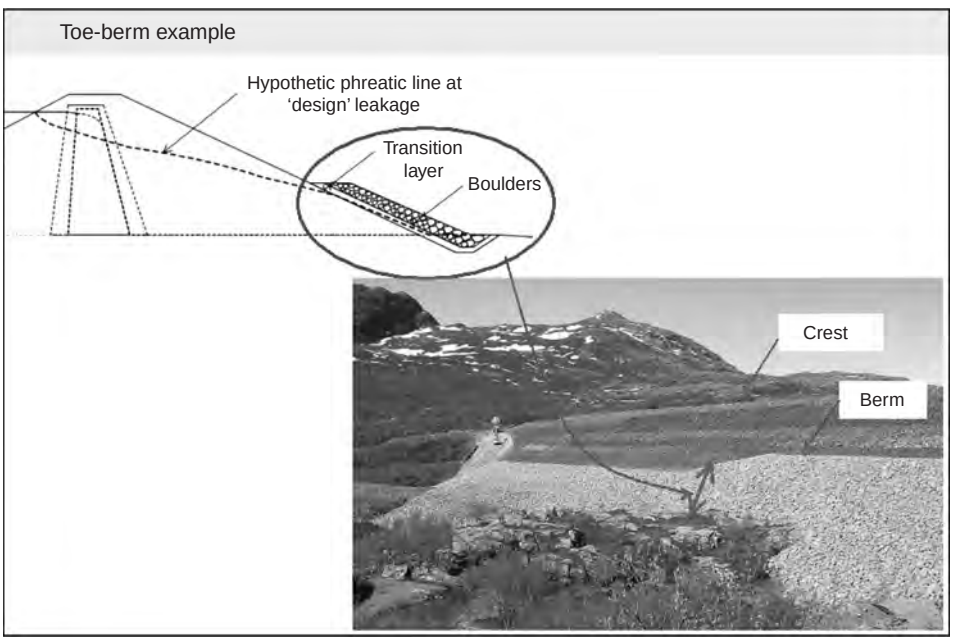


Figure 10.57 Example of a part height "Swedish berm" (Nilsson, 2009).

In Norway the regulator has some minimum requirements for the through flow for which the berm should be designed. These are described in NVE (2012). In summary they are that a total design leakage of 5 m^3/sec or a flow per metre length of dam of 0.5 m^3/sec is required. The size of the rockfill in the berm is determined by the methods outlined in Section 8.11.4.

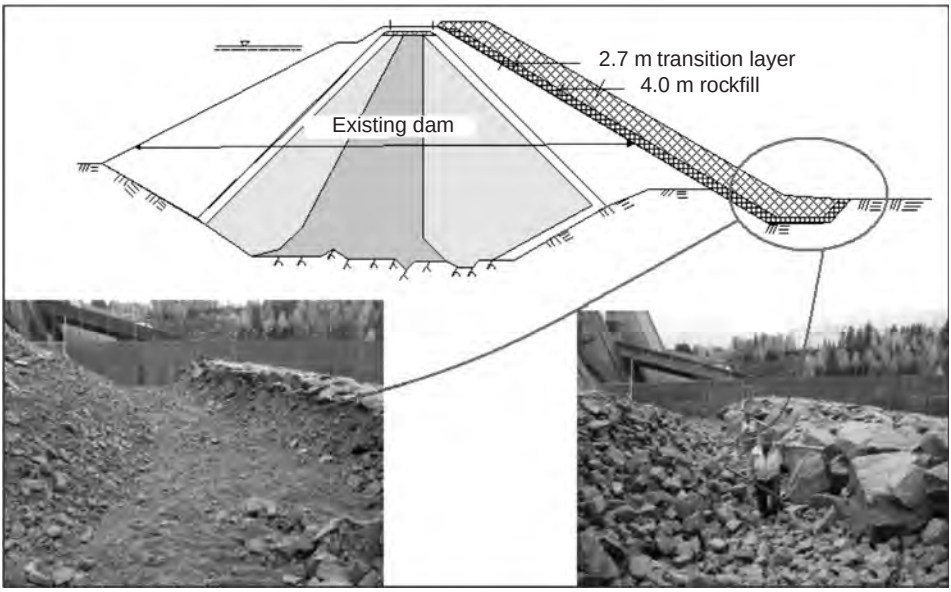


Figure 10.58 Example of a full height "Swedish berm" (Nilsson, 2009).

The principle of the berms is that they will control the breach process until the filters seal with material transported by the leakage flow from upstream zones of the dam as has happened in many incidents.

As shown in Figures 10.57 and 10.58 the berm may be full height or only at the toe depending on where internal erosion and piping may be expected to occur and the nature of the existing rockfill.

Nilsson (2013) advises that a transition layer is provided between the existing fill and the rockfill berm. This is designed to allow the fines from the core and the existing fill to pass so there is not a build up of pressure on the interface. This will result in higher permeability existing fill which needs to be allowed for in designing the rockfill in the berm.

If sealing on the transition is likely then the berm must be designed accordingly with higher pore pressures.

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Chapter 11

Analysis of stability and deformations

11.1 ANALYSIS OF STABILITY AND DEFORMATIONS METHODS OF ANALYSIS

The analysis of the stability of dams and other slopes is usually carried out using Limit Equilibrium Analysis (LEA). In most situations more detailed analysis using numerical methods, e.g. finite element analysis, is not necessary. However if it is necessary to model the deformations and stresses of the dam or the effects of strain weakening, as in progressive failure, numerical methods must be used. Comprehensive reviews of slope stability analysis have been written by Fredlund and Krahn (1977), Whitman and Bailey (1967), Fredlund (1984), Graham (1984), Mostyn and Small (1987), Duncan (1992, 1996a and b), Morgenstern (1992), and Hungr (1997).

Whether carrying out limit equilibrium or numerical analysis, it is important to formulate the problem correctly and, in particular, to establish whether undrained strengths (total stress analysis) or drained strengths (effective stress analysis) is used. Table 11.1 summarises the shear strengths, pore pressures and unit weights which should be used.

The reason for using total stress analysis (Su, ϕ_u) in low permeability zones for end of construction, rapid drawdown and staged construction condition is that it is difficult to predict the pore pressures accurately. Undrained strengths should also be used for contractive soils as described in Section 6.1.3.

If in doubt as to whether undrained or drained conditions will be most critical, it is wise to check both and adopt the results from the lowest strength case. Undrained conditions can occur even in relatively slow rates of change of reservoir levels. Cooper *et al.* (1997) describe a situation where annual cycles of rise of reservoir level were sufficient to induce undrained loading conditions in the downstream poorly compacted (contractive) clay zone of the concrete core wall Embankment No. 1 at Hume Dam. This is described in Section 11.6.1.

Effective stress methods can be used for the end of construction and have the advantage that monitoring of pore pressures can assist in assessing the stability. Section 11.4.2 describes the methods for estimating the pore pressures in the partially saturated soil as it is compacted in the dam and loaded by the weight of the embankment as construction continues.

It is important to model the foundation of the dam in the analysis of stability, particularly where the dam is founded on soil or on rock with low strength surfaces

Table 11.1 Shear strengths, pore pressures and unit weights for stability analysis (adapted from Duncan 1992).

	Condition		
	End of construction	*Rapid draw-down and staged construction*	*Normal operating ('steady seepage')*
Analysis procedure and shear strength for free draining zones – filters, rockfill, sand/gravel in foundations	Effective stress analysis, using c', ϕ'	Effective stress analysis, using c', ϕ'	Effective stress analysis, using c', ϕ'
Analysis procedure and shear strength for low permeability zones	Total stress analysis using Su and ϕ_u[(1)] or effective stress analysis modelling partially saturated conditions	Total stress analysis using Su and ϕ_u[(1)] for the dam prior to draw down or construction of the second stage	Effective stress analysis using c', ϕ', unless soils are contractive[(2)] in which case use Su measured in the dam.
Internal pore pressures	No internal pore pressures (u) for total stress analysis; set u equal to zero in these zones. Pore pressures determined from laboratory tests for effective stress analysis	No internal pore pressures (u) for total stress analysis; set u equal to zero in these zones. Pore pressures from seepage analysis for effective stress analysis	Pore pressures from seepage analysis and/or from piezometer readings for effective stress analysis.
Reservoir water	Include (usually as a zone with $c' = 0$, $\phi' = 0$, $\gamma = 9.8\,kN/m^3$)	Include (usually as a zone with $c' = 0$, $\phi' = 0$, $\gamma = 9.8\,kN/m^3$)	Include (usually as a zone with $c' = 0, \phi' = 0$, $\gamma = 9.8\,kN/m^3$)
Unit weights[(3)]	Total	Total	Total

Notes:
(1) Su and ϕ_u describe the undrained strength envelope, so the variation in undrained strength, with increase in total stress, can be modelled in the analysis.
(2) Contractive soils include poorly compacted saturated clay fill, normally and lightly over-consolidated clays, and other situations described in Section 6.1.3. Effective stress analysis which ignores pore pressures generated on shearing over-estimate the factor of safety.
(3) For free draining zones use γ_{dry} or γ_{moist} for zones above water, γ_{sat} below. For low permeability zones, use γ_{sat} or γ_{moist}.

within it, e.g. bedding surface shears. Most failures of dams have occurred where such weak zones in the foundation have not been recognised.

11.2 LIMIT EQUILIBRIUM ANALYSIS METHODS

11.2.1 General characteristics

Table 11.2 summaries the characteristics of the methods of slope stability analysis.

These methods all employ the same definition of the factor of safety, F:

$$F = \frac{\text{Shear Strength of the Soil}}{\text{Shear Stress required for equilibrium}}$$

Table 11.2 Characteristics of limit equilibrium methods of slope stability analysis (adapted from Duncan 1992 after Duncan and Wright, 1980).

Method	*Characteristics*
Slope Stability Charts (Janbu, 1968; Duncan, *et al*., 1987)	Accurate enough for some purposes for initial estimates. Faster than detailed computer analyses
Ordinary Method of Slices (Fellenius, 1927)	Only for circular slip surfaces Satisfies moment equilibrium Does not satisfy horizontal or vertical force equilibrium Underestimates the factor of safety in most cases
Bishop's Modified Method (Bishop, 1955)	Only for circular slip surfaces Satisfies moment equilibrium Satisfies vertical force equilibrium Does not satisfy horizontal force equilibrium
Force Equilibrium Methods (e.g. Lowe and Karafiath, 1960, and U.S. Corps of Engineers, 1970)	Any shape of slip surfaces Do not satisfy moment equilibrium Satisfies both vertical and horizontal force equilibrium
Janbu's Generalised Procedure of Slices (Janbu, 1968)	Any shape of slip surfaces Satisfies all conditions of equilibrium Permits side force locations to be varied More frequent numerical problems than some other methods
Morgenstern and Price's Method (Morgenstern and Price, 1965)	Any shape of slip surfaces Satisfies all conditions of equilibrium Permits side force orientations to be varied
Spencer's Method (Spencer, 1967)	Any shape of slip surfaces Satisfies all conditions of equilibrium Side force are assumed to be parallel

The analyses assume the factor of safety is uniform along the whole of the failure surface and cannot directly allow for localised strain weakening or progressive failure effects.

The number of equations of equilibrium available is smaller than the number of unknowns in limit equilibrium analysis, so assumptions are made to make the problem determinate. This is what differentiates most of the methods in Table 11.2. Of the methods listed, the ordinary (or Fellenius, 1927) method should not be used, because it generally underestimates the factor of safety. In force equilibrium methods the factor of safety is affected significantly by the assumed inclination of the side forces between slices and are therefore potentially not as accurate.

Most practitioners use the Bishop 'modified' (or 'simplified') method for circular analysis, which has been shown to be adequately accurate (Whitman and Bailey, 1967) and is stable computationally, but most methods give similar answers. For non-circular surfaces, the Morgenstern and Price (1965) and Spencer (1967) methods are widely used.

Hungr (1997) has demonstrated that for non-circular surfaces the methods may give different answers. The example is a simple bi-planar sliding block (Figure 11.1), defined by a flat basal surface (Plane A) and a steeper back-scarp (Plane B). Either of the two planes can be a weak surface. The second plane may be a stronger structural feature or it may be a rupture plane extending through intact material. A parametric

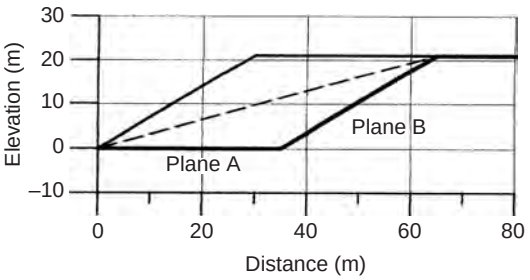


Figure 11.1 A bi-linear rupture surface used in the parametric study to compare methods of stability analysis. The dashed line is the piezometric surface (Hungr, 1997).

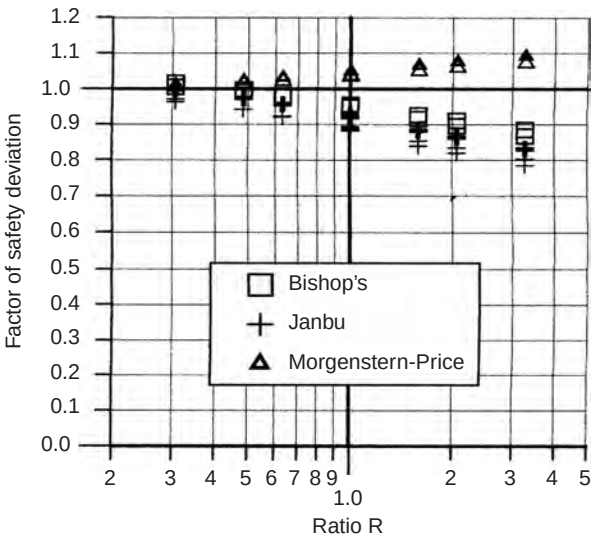


Figure 11.2 Comparison of the factors of safety obtained by four methods of analysis, for the bi-linear surface shown in Figure 11.1. The factors are normalized with respect to the Spencer's Method. The abscissa is the ratio of strengths available on the two parts of the rupture surface (Hungr, 1997).

study was carried out, varying the friction angle on Planes A and B between 10°, 20° and 30° with no cohesion. Both dry conditions and pore-pressures determined by the piezometric surface shown in Figure 11.1 were examined. The resulting Factors of Safety ranged between 0.5 and 2.5 for the range of conditions.

Figure 11.2 shows a comparison between the different methods of analysis. The abscissa in this figure is the ratio, R, between the friction coefficient on Plane A and that on Plane B: ϕ

$$R = \frac{\tan \varphi'_A}{\tan \varphi'_B} \tag{11.1}$$

The ordinate is a ratio between Factors of Safety determined by the various methods and that obtained by Spencer's Method, which is used as a reference. All the methods are shown to converge to the same value when R is low, i.e. when the flat basal plane is weaker than the back scarp. On the other hand, the methods diverge by as much as 30% in the case when the back scarp is weak and the sliding body is supported by the toe plane. Under these conditions there is a significant difference even between Spencer's and Morgenstern-Price methods.

Janbu's Simplified method produces a Factor of Safety which is consistently less than that of Bishop's (with the Fredlund and Krahn modification). The Janbu correction factor is required to compensate for this. All of these trends persist in equal measure for dry conditions and with pore-pressure.

In practice, cases where R is low are fortunately more frequent than the opposite, as Plane A often follows a thin weak layer in the stratigraphy, a bedding plane or a near-horizontal pre-sheared surface. Under such conditions there is not much difference between the four methods and the user is therefore justified in taking advantage of the high efficiency of Bishop's Simplified Method in two or three dimensions.

On the other hand, in cases where the strongest element of the sliding surface is at the toe, rigorous methods such as Morgenstern-Price should be used. Examples of this are cases where the back scarp (Plane B) follows steeply inclined bedding, a fault surface or similar and the sliding body is supported by a strong toe.

11.2.2 Some common problems

Some common problems in LEA (which are often simply omissions and/or occur through lack of proper care) include:

- Use of circular rather than non circular analysis, where failure surfaces will clearly follow weak surfaces – e.g. bedding surface shears. Figure 11.3 shows examples where non circular analysis must be used.
- Omitting to model cracks in the ground surface and the water pressure which will often develop in the cracks due to rainfall or surface water flowing into the crack. This can have a significant effect on calculated factors of safety, particularly for smaller failure surfaces near a dam crest.
- Not modelling anisotropy of strengths, e.g. in fissured clays or stratified soil and rock.
- Not clearly showing on the drawing of the cross section and failure surface the properties used for that particular analysis. If the two are separated confusion is likely to occur.

These features can all be modelled with modern computer programs. If the program being used cannot model these features, consideration should be given to purchasing a program which can handle these issues.

A more subtle problem, which is an extension of the discussion on the different methods of calculation described above, is where essentially translational sliding in rock is being modelled. Figure 11.4 shows such a slope, where the rock is relatively strong and with a few, widely spaced, near vertical joints AB, CD, EF (e.g. sandstone or conglomerate), with potential sliding on a weak layer (e.g. claystone) OBDF.

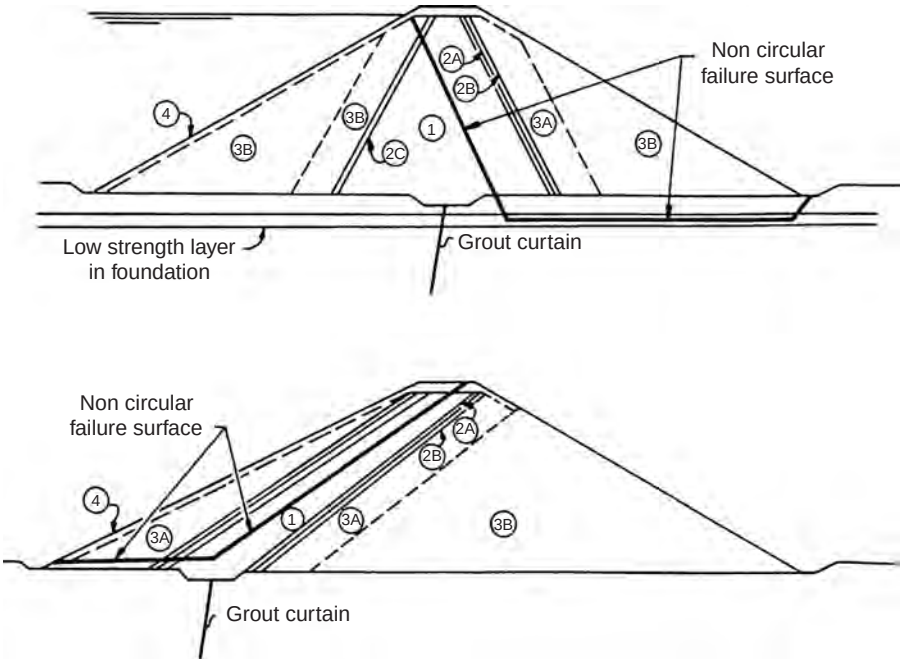


Figure 11.3 Examples where non circular failure surfaces must be analysed.

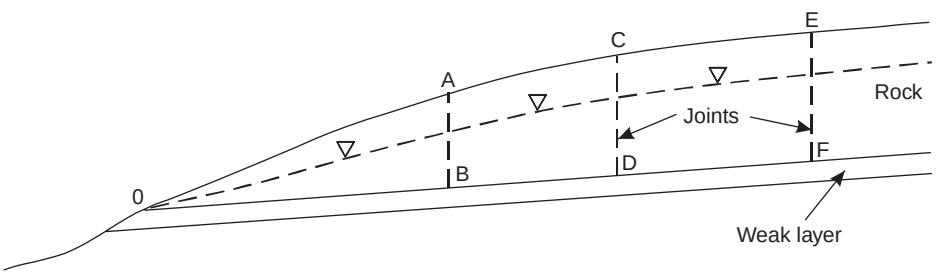


Figure 11.4 Limit equilibrium analysis of a rock slope sliding on a weak layer.

The driving force for instability is the groundwater in the joints and the gravity forces along OBDF.

The use of a conventional computer program to analyse the stability of OAB, OCD, or OEF will overestimate the factor of safety substantially even if the failure surfaces are pre-defined, because if one inserts a reasonable joint strength (say $c' = 0$, $\phi' = 40°$), the program assumes this applies to the rock and assumes slice slide forces will apply, equivalent to say a wedge of 'rock' with $c' = 0$, $\phi' = 40°$ which will also drive the instability. This is difficult to overcome in computer programs other than modelling the vertical part of the failure surface (e.g. EF) as a crack. It is also possible

this type of problem will lead to convergence problems in the non-circular analysis. It is better to analyse such cases using a wedge analysis (either by hand or a computer wedge analysis).

Other problems which arise are (Duncan 1992):

- Failure of the person performing the analysis to understand soil mechanics well enough to know how to define the water pressures, unit weights and shear strengths appropriate for the analysis.
- Failure of the person performing the analysis to understand the computer program well enough to define these quantities correctly in the input.
- Failure of the person performing the analysis and the person reviewing the results to check the results and properly evaluate their reasonableness.

11.2.3 Three dimensional analysis

The status of three-dimensional analysis is summarised in Duncan (1992), Morgenstern (1995) and Fell *et al.* (2000). The Bishop's Simplified, Janbu and Spencer's Methods have been extended into three dimensions (Hungr *et al.*, 1989, Lam and Fredlund, 1993). However, none of the available algorithms can account for the internal stresses existing in a non-rotational and laterally asymmetric problem (Hungr, 1994). A true 'rigorous method' that would satisfy all the available equilibrium conditions has not yet been formulated. As suggested by Hungr (1994), a rigorous three-dimensional method would require the definition of five spatially distributed inter-column force functions. The solution for the Factor of Safety and five inter-column force inclination ('i') coefficients would require six nested levels of iteration. While modern computers could easily handle the numerical computation, the correct selection of the inter-column force functions and possible convergence difficulties pose a daunting research problem.

The authors experience in the use of 3D analyses has been that the users have had a learning curve to get the programs to work satisfactorily. The methods are also limited where there are thin weaker layers, e.g. within embankment fill as shown in Figure 11.5 because the programs set out to model curved surfaces and cannot follow the weak layer satisfactorily.

As summarized by Fell *et al.* (2000) the use of 'rules of thumb' such as a 10% increase to compensate for the neglect of 3D effects, as suggested in some textbooks, is not advisable, because although, as proven by Cavounidis (1987), the Factor of Safety of the critical 3D sliding surface always exceeds the critical 2D factor, the ratio between the two can vary within a range of 1.0 to as high as 1.4 (Morgenstern, 1992; Hungr *et al.*, 1989).

Potential problems lie in situations where the stability analysis is used to estimate the strength of certain materials through back analysis. Neglecting a strong 3D effect in the back-analysis could result in a serious overestimation of the back-calculated strength. However provided that the geometry of the failure surface remains the same for the back-analysis and the 'forward analysis,' this is not a major practical problem.

The authors have on occasions resorted to, or have seen others resort to, reliance on three-dimensional effects to show that a factor of safety is adequate. In most cases this has been done simply by averaging the factors of safety of sections taken adjacent

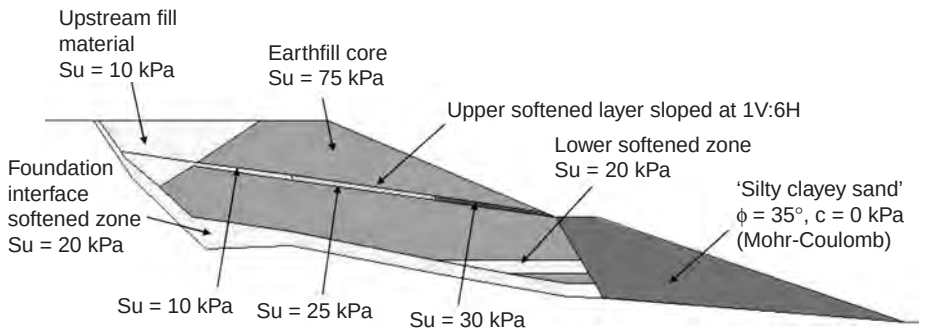


Figure 11.5 Section through Cairn Curran Dam showing thin softened layers at the foundation/fill contact and within the fill (Swindon *et al.*, 2004).

to each other. Usually, if the situation is so marginally stable, it is in reality almost certainly in need of some remedial action to improve the factor of safety.

11.2.4 Shear strength of partially saturated soils

In general, unsaturated soils have a shear strength that is larger than that of the same soil saturated because the negative pore-water pressure within the soil (known as the matric suction) increases the effective stress. Laboratory experiments have shown that the shear strength may not change at the same rate as for total and positive pore water changes but at a lower rate.

The most widely used approach for modelling the shear strength of partially saturated soils. in slope stability programs is that of Fredlund *et al.* (1978). They proposed that the shear strength could be expressed as a linear combination of the net normal stress $(\sigma_n - u_a)$ and of the matric suction $(u_a - u_w)$:

$$\tau_f = c' + (u_a - u_w)\tan\phi^b + (\sigma_n - u_a)\tan\phi \tag{11.2}$$

where:
c' = effective cohesion (the same as for the saturated soil)
ϕ' = effective angle of internal friction (the same as for the saturated soil)
ϕ^b = an angle defining the increase in shear strength for an increase in matric suction.
σ_n = total normal stress
u_w = pore water pressure
u_a = pore air pressure
$(\sigma_n - u_a)$ = net normal stress
$(u_a - u_w)$ = matric suction

As discussed in Fredlund *et al.* (1987) the partially saturated shear strength envelope is non-linear, with ϕ^b approaching ϕ' at high degrees of saturation. Fredlund *et al.* (1978) and Fredlund and Rahardjo (1994) describe how the strengths can be determined in laboratory tests.

Fredlund and Rahardjo (1994) present some test data from several authors which showed a range of ϕ^b from 13 to 22 degrees and $(\phi' - \phi^b)$ from 5 to 21 degrees with larger values applying to soils with high ϕ' values. The SLOPE W manual suggests most ϕ^b values are between 15 and 20 degrees.

The matric suction should for the soil be determined from filter paper of pressure plate techniques which give the soil water characteristic curve.

Another approach for the determination of the shear strength of unsaturated soils is described in Fleureau *et al.* (1995); Oberg and Sallfors (1995); Bolzon *et al.* (1996) and Khalili and Khabbaz (1998). In this approach the shear strength is determined on the basis of the effective strength parameters c′ and ϕ' and a single stress variable defined as:

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \tag{11.3}$$

in which σ' is the effective stress and χ is the effective stress parameter, which has a value of 1 for saturated soils and a value of 0 for dry soils. The advantage of the effective stress approach is that the change in the shear strength with changes in total stress, pore water pressure and pore air pressure is related to a single stress variable. As a result a complete characterisation of the soil strength requires matching of a single stress history rather than two or three independent stress variables. Furthermore, the approach requires very limited testing of soils in an unsaturated state. A major difficulty of the effective stress approach has been the determination of χ.

Khalili and Khabbaz (1998) used experimental data to demonstrate that χ may be determined from the using the equation

$$\chi = \left[\frac{(u_a - u_w)}{(u_a - u_w)_b}\right]^{-0.55} \tag{11.4}$$

where $(u_a - u_w)_b$ is the air entry value and

$\dfrac{u_a - u_w}{(u_a - u_w)_b}$ is termed the suction ratio.

The air entry value, also called the bubbling pressure, corresponds to the matric suction above which air penetrates into the soil pores and can be measured using the filter paper technique (ASTM, 1992; Swarbrick, 1992, 1995) or pressure plate technique (ASTM, 1968, 1992).

This approach was further assessed by Geiser (2000) who demonstrated an even stronger correlation between χ and the suction ratio than Khalili and Khabbaz (1998) had with their data.

In practice most stability analyses carried out ignore the effects of partial saturation, but in so doing, the factor of safety, particularly for the upper part of the dam, may be significantly underestimated (i.e. give a conservative estimate of the FOS). Hence if this is shown to be critical, either an approach such as that of Fredlund *et al.* (1978) or Khalili and Khabbaz (1998) should be used to estimate the strength above the full supply level (or above the phreatic surface) or alternatively undrained strength

estimated for existing dams by in-situ testing such as CPT and/or laboratory tests on samples tested at the *in situ* moisture content.

However it is essential that allowance be made for potential cracking or softening in areas affected by cracking as discussed in Section 11.3.2.

A widely used stability analysis program Slope W has built in to it the ability to model partial saturation using the Fredlund *et al*. (1978) approach. This requires users to enter ϕ^b values which can be assessed as discussed above. To carry out conventional stability analyses where suction effects are ignored ϕ^b should be set as zero.

11.3 SELECTION OF SHEAR STRENGTH FOR DESIGN

11.3.1 Drained, effective stress parameters

11.3.1.1 *Peak, residual or fully softened strength in clay soils?*

Whether peak, softened or residual strengths are used in the analysis of slope stability depends on the presence or absence of existing slide 'planes' (actually surfaces which may not be truly planar) and fissuring. The following guidelines are given:

- Where there is an existing slide plane, e.g. in the foundation of a dam or where a dam has failed by sliding, the field residual strength should be used for the slide plane. This applies regardless of how long it has been since sliding last occurred. The exception could be where there is definite evidence of re-cementing, e.g. in some slickensided joints in weathered rocks.
- Bedding surface shears formed by folding of rock strata have strengths approaching residual and, unless extensive tests on the surfaces show otherwise, residual strengths should be used.
- Compacted soils and soils which have no fissuring or cracks should be assigned peak strength parameters c′ and ϕ' determined from saturated remoulded samples compacted to the same density ratio and moisture content the soil would be in the dam. The majority of dam embankment design is therefore based on peak strength parameters. However if the dam has experienced significant deformations of the upstream slope under drawdown conditions it is likely that the strains have localised to a degree and the soil dilated sufficiently to where fully softened strengths may be more applicable.
- Cuts in soils are often designed using fully softened strength to allow for the softening due to the unloading of the slope and some progressive failure mechanisms which may be present.
- Fissured soils have strength between peak and residual strength depending on the nature of fissuring, orientation, continuity and spacing of the fissures. Such soils do occasionally occur in dam foundations, e.g. Ross River Dam (McConnel, 1987) and Prospect Dam. This is discussed in more detail in Section 6.1.8. Soils with relict joints, such as extremely weathered basalt, behave in a similar manner.
- Triaxial and direct shear tests do not properly simulate the actual plane strain stress conditions which exist in most slope stability problems. However, the uncertainty arising from this is not significant in practical slope stability problems and, provided the tests are carried out and interpreted correctly, the results from both tests can be adopted for design.

It should be noted that, as pointed out by Lade (1986) and Mitchell (1993), apart from cemented soils the effective cohesion (c′) should be zero or very small. The value of c′ adopted is particularly critical in the analysis of the stability of smaller dams; the upper parts of larger dams and in smaller landslides, because it has a major effect on calculated factors of safety when failure surfaces are shallow. It is recommended that unless there is definite evidence of higher values, the effective cohesion adopted for design should be between 0 and 10 kPa for peak strength and should be zero or say 1 kPa for residual and softened strengths, at least at low normal stresses. In some situations where the strength envelope is markedly curved, a bi-linear envelope such as that shown in Figure 11.22 may be used.

11.3.1.2 *Selection of design parameters in clay soils*

When several triaxial tests have been carried out on the one soil, it is recommended that the design shear strength parameters are obtained from a p′-q plot of the test results, rather than by, say, averaging the individual c′, ϕ' values from each test, or plotting all Mohr's circles on one diagram. It is also important to use results from the effective stress range applicable to the field problem. In many cases this is at low stress, e.g. less than 50 kPa to 100 kPa for slide surfaces at 5 m depth.

The p′-q plot is a graph of the apex points of the Mohr's circles from the test results as shown in Figure 11.6.

When selecting the design parameters for design of dam embankments and landslide stabilizing works, it is common to bias towards the conservative by selecting a line with, say, 75% of the test points above and 25% below (i.e. a lower quartile line), but a line of best fit or a lower bound may be adopted depending on the circumstances.

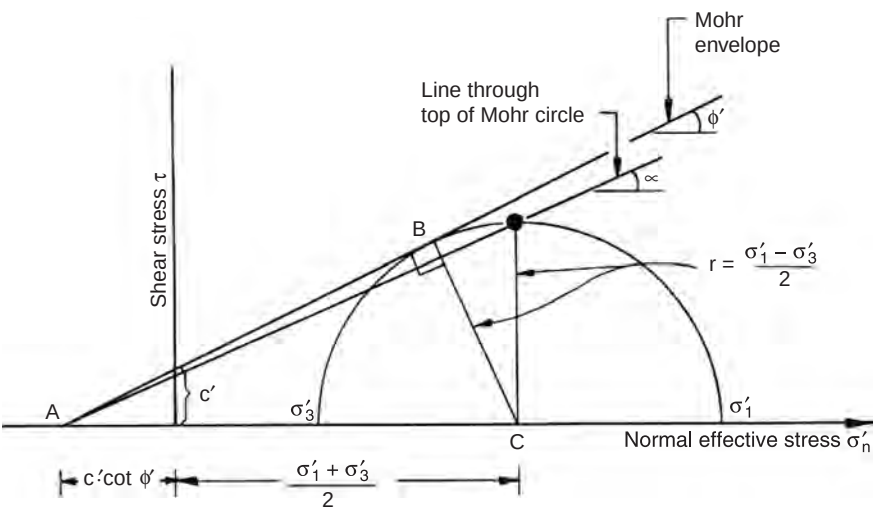


Figure 11.6 Basis of p′-q plot where $p' = (\sigma'_1 + \sigma'_3)/2$, $q = (\sigma'_1 - \sigma'_3)/2$; it can be shown that $\phi' = \sin^{-1}(\tan\alpha)$; $c' = a/\cos\phi'$; where a and α are obtained from the p′-q plot as shown on Figure 11.7.

If direct shear tests have been used to determine strength parameters the results should be plotted on a graph of normal stress vs shear stress and design parameters selected by a similar procedure to that outlined below for triaxial tests.

In any slope stability analysis it is good practice to check the calculated factor of safety for a range of strengths, e.g. lower quartile and lower bound, to determine the sensitivity of the factor of safety to the assumed strength.

It is essential to use judgement in selecting the design line, rather than using say a least squares regression analysis, as judgement enables allowance to be made for:

- Poor individual test results.
- The general trend for the second and particularly third stage of staged tests to give a lower strength than the true one because of excessive deformation and some loss of strength from the peak due to displacement on the shear plane, particularly in sensitive clays or cemented soils.
- General curvature of the Mohr's circle envelope.
- Adoption of low effective cohesion (c′) which correctly models the behaviour of most soils.

The average of the c′ and ϕ' values from which the p′-q plot was derived, is also shown in Figure 11.7. It can be seen that using averages tends to give a larger c′ and lower ϕ', than using the p′-q diagram. This is generally non-conservative for smaller dams, shallow failure surfaces in larger dams and landslide stability, because the strength is overestimated in the working stress range.

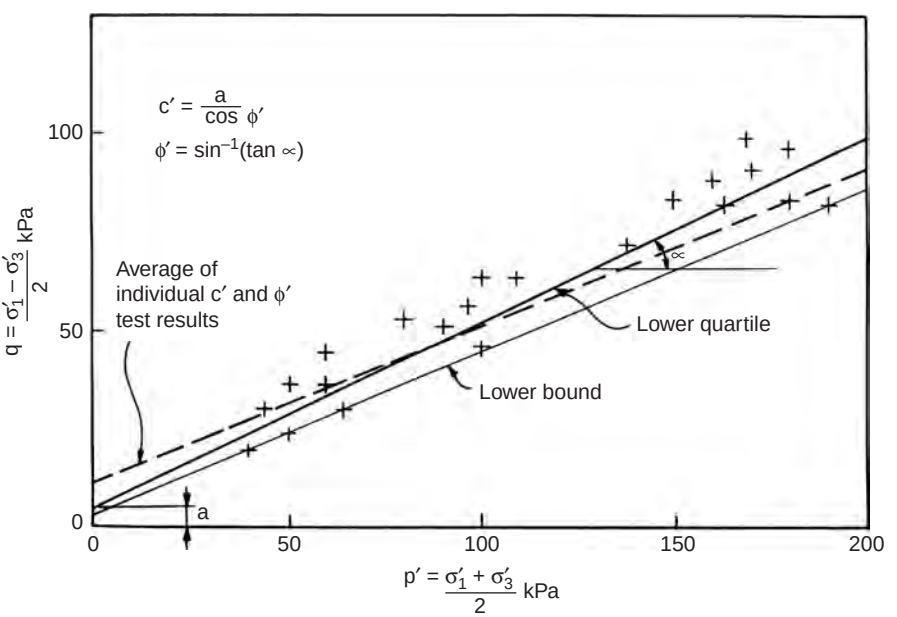


Figure 11.7 Typical p′-q plot of triaxial test results.

Wroth and Houlsby (1985) have suggested a method based on the critical state parameters. This method is similar in principle to that proposed above using a p′-q diagram, but takes account of the variation of c′ with water content at failure. The critical P, Q values ($P = (\sigma' + 2\sigma'_3)/3$ and $Q = (\sigma'_1 - \sigma'_3)/2$ at failure are normalised by dividing by the equivalent pressure at that water content (the pressure on the normally consolidated line at the same voids ratio as the specimen). Wroth and Houlsby claim that this gives a more rational way of selecting c′ as a function of water content, particularly for fills. In practice this degree of sophistication may not be warranted.

11.3.1.3 *Selection of design parameters – granular soils and rockfill*

The strengths of granular soils and rockfill should be selected to account for the confining pressure, degree of compaction and particle size distribution and, for rockfill, the rock substance strength and finer particles content as described in Section 6.2.

In most cases, particularly for rockfill, a bi-linear strength envelope should be used to model properly the curvature of the strength envelope. If this is not done, the factor of safety of smaller failure surfaces, particularly at the crest of the dam, will be significantly under-estimated.

Depending on the confidence with which the strengths are able to be estimated (reflecting whether there are good records of how the dam was built or not and whether testing was carried out), either median or lower quartile strengths should be adopted.

11.3.2 Undrained, total stress parameters

11.3.2.1 *Triaxial compression, extension or direct simple shear strength*

As discussed in Section 6.1.3 and shown in Figures 6.11 and 6.12, it is important to allow for the stress path in selecting the undrained strength. The undrained strength estimated from triaxial compression tests only may significantly over-estimate the strength.

11.3.2.2 *Selection of design parameters*

The undrained strength of natural soft clay deposits usually shows a considerable scatter which is due to natural stratification of the deposit and measurement errors. Figures 11.8 and 11.9 show typical soft ground strength profiles with an overconsolidated crust over the near normally consolidated soil.

In these circumstances it is recommended that where the data plots as in Figure 11.8 the median strength be used after discarding outlying results but, where there is considerable scatter as in Figure 11.9, a more conservative e.g. lower quartile strength be adopted.

In the overconsolidated crust Ferkh and Fell (1994) adopted an empirical approach, using the median strength line – which uses a strength equal to the median strength up to where the liquidity index, LI, = (water content – plasticity index)/(liquid limit – plastic limit) – is less than 1.0. These guidelines were developed for soft clay deposits, which are not usually found where dams are being built, but the principles are applicable. Rather than use the empirical approach detailed above it would be better to assess the fissured strength for the overconsolidated crust, as detailed in Section 6.1.8.

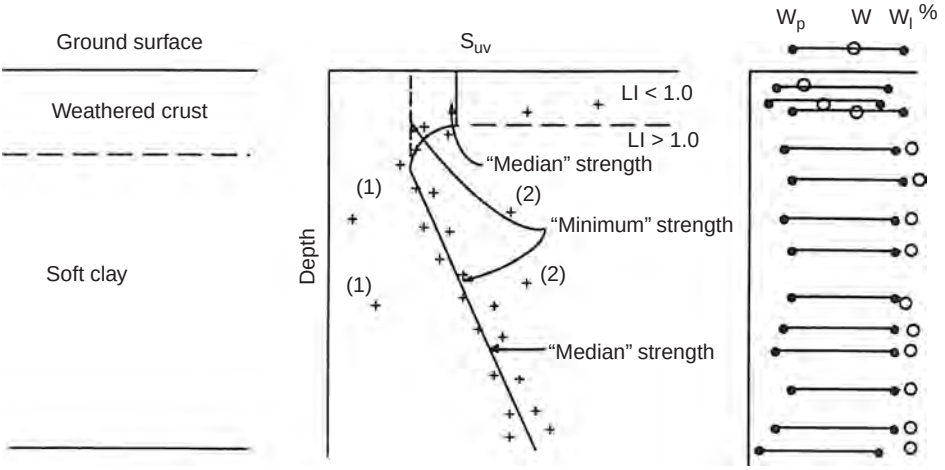


Figure 11.8 Selection of foundation strength – Normal case. Notes: (1) Discard, probably disturbed. (2) Discard, probably affected by shells, roots etc. (Ferkh and Fell, 1994).

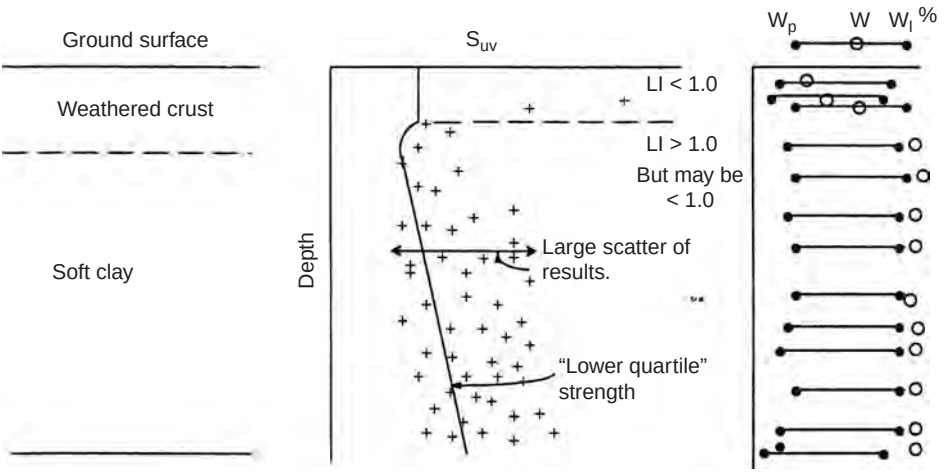


Figure 11.9 Selection of foundation strength – influenced by shells, roots etc. and large scatter of data (Ferkh and Fell, 1994).

For embankments:

(a) The undrained strength varies within and between the layers of the compacted fill, reflecting varying degrees of softening as the soil wets up with seepage from the reservoir. This is demonstrated in Figure 11.9 with the variation in CPT cone resistance. Note that $Su = (q_c - \sigma_{vo})/N_K$ (see Section 6.1.7.1).

(b) The earthfill has significant undrained strength at the surface, because the soil is compacted in a partially saturated condition. In Figure 11.10 the strength is

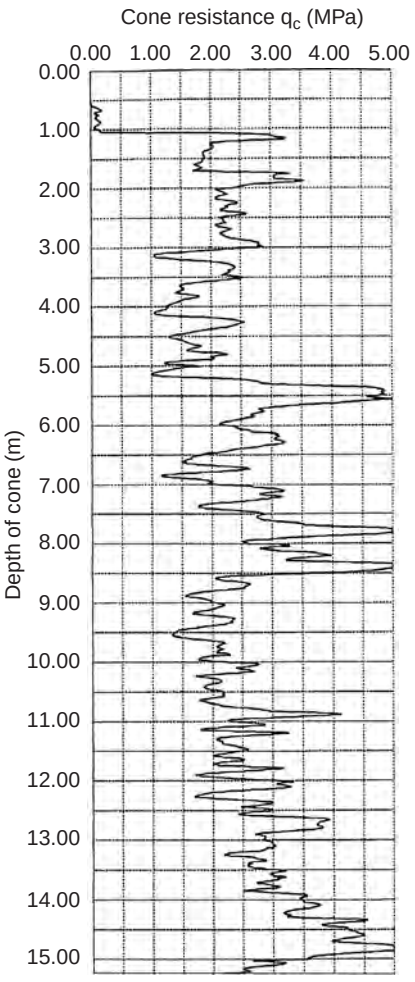


Figure 11.10 Cone Resistance q_c determined from Cone Penetrating Test, versus depth below dam crest for Eppalock Dam (Courtesy Goulburn Murray Water and Snowy Mountains Engineering Corporation).

somewhat less above Full Supply Level (the top 5 m approx.) due to the effects of cracking and softening as water has entered the cracks. This is not an uncommon feature particularly for central core earth and rockfill dams with poorly compacted rockfill and is due to spreading of the dam as it settles as shown in Figures 11.11 and 11.12.

The selection of undrained strength and stability analysis should allow for these features and may include:

– Allowance for cracks, or softened zones around old cracks with little or no shear strength. These are unlikely to persist below FSL.

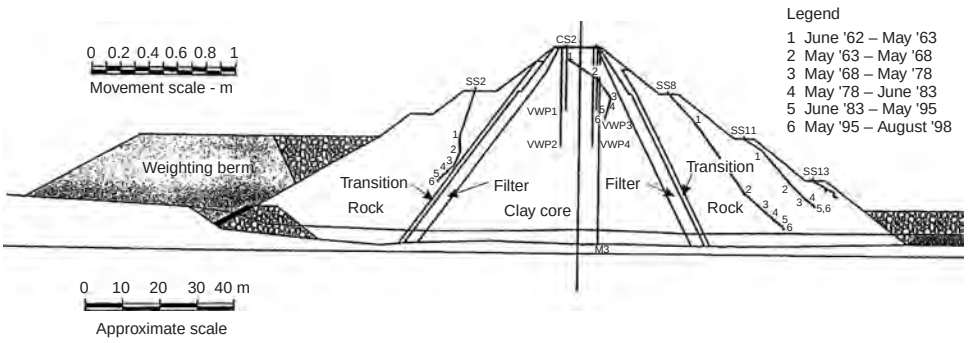


Figure 11.11 Deformation of Eppalock Dam (Courtesy of Goulburn Murray Water and URS Corporation).



Figure 11.12 Cracking and softening due to settlement and lateral spreading at the crest of an embankment dam (Courtesy Goulburn Murray Water and URS Corporation).

- Allowance for water pressures in these cracks.
- Averaging of the strength for failure surfaces which will pass through many layers of the compacted soil, i.e. steeply sloping failure surfaces.
- Adoption of the strength of lower strength layers for failure surfaces which are horizontal. However care needs to be taken in this, since it is not usually likely

Table 11.3 Coefficient of variation (COV) for available data on soil properties (Kulhawy, 1992, from Kulhawy *et al.*, 1991).

Property		*No. of Studies*	*Mean COV without outliers (%)*
Index	– Natural water content, w_n	18	17.7
	– liquid limit, w_L	28	11.3
	– plastic limit, w_P	27	11.3
	– initial void ratio, e_i	14	19.8
	– unit weight, γ	12	7.1
Performance	– effective stress friction angle, ϕ'	20	12.6
	– tangent of ϕ'	7	11.3
	– undrained shear strength, s_u	38	33.8
	– compression index, C_c	8	37.0

that weak layers will persist across the zone, unless they are due to desiccation or freezing in a prolonged shut-down during construction or a particularly poor period in compaction control.

11.3.3 Inherent soil variability

When assessing the strength of materials for stability analysis it should be recognised that there are inherent uncertainties in the strengths of the soil and there are potential errors in the measurement of the strength and the formulae used to transform the measured value (e.g. SPT 'N' value or CPT q_c value) to strength. This is discussed in Kulhawy (1992).

Tables 11.3 and 11.4 present the coefficient of variation (COV = standard deviation/mean) in soil properties and the variability in *in situ* tests, excluding outliers in the data.

It should be noted that there is an inherent greater uncertainty in the undrained shear strength than in the effective shear strength. As discussed below, this should be reflected in the factor of safety used for undrained (total stress) analyses. The test variability shown in Table 11.4 shows a greater uncertainty in SPT tests, than in CPT tests.

However it must be remembered that there is considerable further model uncertainty in the assessment of strengths (and other properties) from the SPT, CPT and Vane Shear – see Sections 6.1.7 and 6.1.9. For example the shear strength of clays can be estimated from the CPT using the equation:

$$Su = \frac{q_c - \sigma_{vo}}{N_k} \tag{11.5}$$

where Su = undrained shear strength; q_c = cone resistance; σ_{vo} = total overburden stress; N_k = Cone Factor.

N_k normally varies from around 10 to 20, but may be higher for overconsolidated soils. Hence the adoption of say $N_k = 15$, as is often done, may introduce an inherent bias of up to 30% or more. These potential errors can and should be reduced by using high quality sampling and laboratory testing to calibrate the *in situ* test for the particular soil and conditions.

Table 11.4 Estimates of *in situ* test variability (Kulhawy, 1992, from Orchant *et al.*, 1988).

Test[a]	COV[b] Equipment	COV (%) Procedure	COV (%) Random	COV[c] (%) Total	COV[d] (%) Range
SPT	5[e] to 75[f]	5[e] to 75[f]	12 to 15	14[e] to 100[f]	15 to 45
MCPT	5	10[g] to 15[h]	10[g] to 15[h]	15[g] to 22[h]	15 to 25
ECPT	3	5	5[g] to 10[h]	7[g] to 12[h]	5 to 15
VST	5	8	10	14	10 to 20
DMT	5	5	8	11	5 to 15
PMT	5	12	10	16	10 to 20[i]
SBPMT	8	15	8	19	15 to 25[i]

a – Test notation: SPT Standard Penetration Test; CPT Cone Penetration Test; VST Vane Shear Test; DMT Dilatometer Test; PMT Pressuremeter Test; SBPMT Self Boring Pressuremeter Test (M = mechanical, E = Electrical).
b – COV = standard deviation/mean.
c – $COV(Total) = [COV(Equipment)^2 + COV(Procedure)^2 + COV(Random)^2]^{½}$
d – Because of limited data and judgment involved in estimating COV, ranges represent probable magnitudes of test measurement error.
e – Best case scenario for SPT test conditions.
f – Worst case scenario for SPT test conditions.
g – Tip resistance, q_c, CPT measurements.
h – Side resistance, fs, CPT measurements.
i – Results may differ for P_o, P_f and P_L, but data are insufficient to clarify this issue.

11.4 ESTIMATION OF PORE PRESSURES AND SELECTION OF STRENGTHS FOR STEADY STATE, CONSTRUCTION AND DRAWDOWN CONDITIONS

11.4.1 Steady state seepage condition

11.4.1.1 *Steady state pore pressures*

Steady state seepage conditions are usually assumed for the assessment of the long term stability of the downstream slope of the dam. These are usually based on the reservoir being at Full Supply Level.

In reality, it may take many years to reach steady state conditions in the dam core. LeBihan and Leroueil (2000) calculate that for typical central core earth and rockfill dams, full saturation will take a few years for a core saturated permeability of 10^{-6} m/sec, several decades for 10^{-7} m/sec and centuries for 10^{-8} m/sec. The authors' experience confirms that large dams may not have reached equilibrium pore pressures 20–30 years after construction, with construction pore pressures dominating in the lower part of the dam and fully saturated conditions not established in the upper parts. It is however good practice to design for the steady state conditions, making appropriately conservative assumptions to estimate the pore pressures.

Pore pressures for the steady state seepage condition are estimated by calculating the flownet for the embankment section either by graphical techniques or most commonly now by finite element methods. These techniques are described in detail in other references, e.g. Cedergren (1967, 1972), Bromhead (1986) and Desai (1975) and are not covered here.

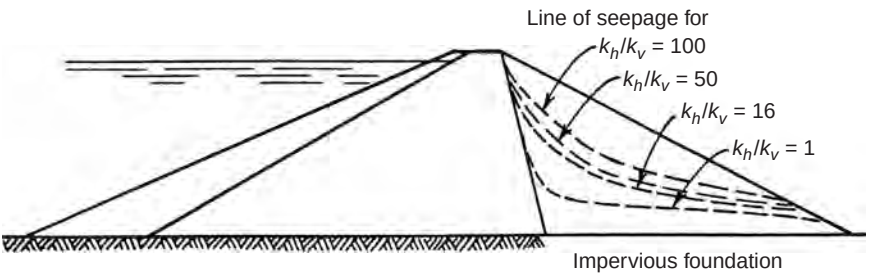


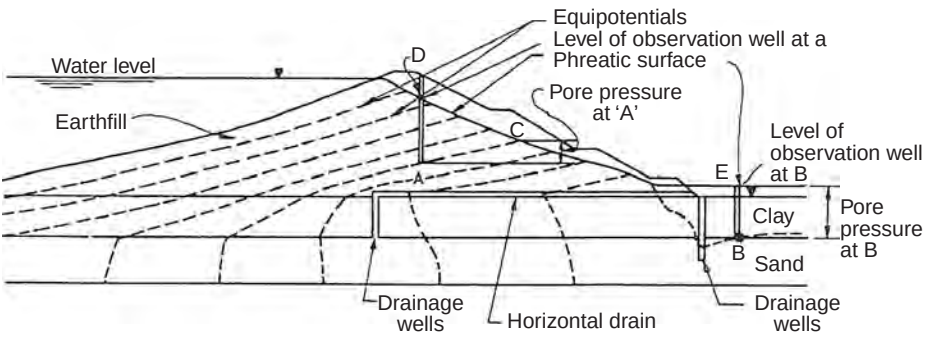
Figure 11.13 Effect of k_H on pore pressures in a zoned earthfill embankment (Cedergren 1972. Reprinted by permission of John Wiley & Sons Inc.).

The following issues are important when calculating the flownet for embankment dams.

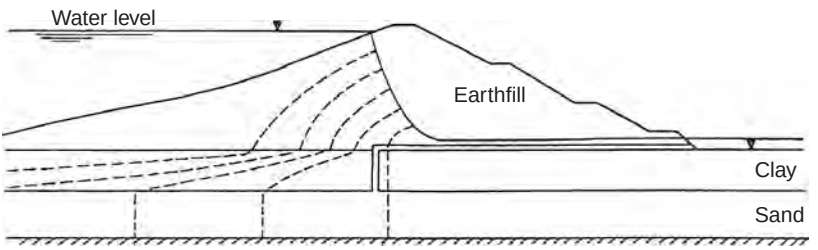
(a) *Zoning of the embankment*. Embankment zoning clearly has a vital role in determining the pore pressures in the embankment. This is discussed in Chapter 8.

(b) *Anisotropic permeability of embankment earthfill*. Earthfill in dam embankments is compacted in layers and the action of rolling, possible drying and cracking of the surface of each layer and greater compaction of the upper part of the layers compared to the lower part will almost invariably lead to the horizontal permeability (k_H) being greater than the vertical permeability (k_V). It would not be unusual for $k_H/k_V = 15$ and even as high as $k_H/k_V \approx 100$. This has a marked effect in pore pressures in an embankment, particularly if no seepage control measures such as a vertical drain are incorporated in the design. Figure 11.13 shows pore pressures for a zoned earthfill dam for $k_H/k_V = 1$, 16, 50 and 100 in the core. The permeability of the downstream zone is 20 times the vertical permeability of the core. It can be seen that pore pressures on the downstream slope are affected greatly by the permeability ratio. The affect on pore pressures of using $k_H/k_V = 15$ and $= 1$ are also evident in Figure 11.14.

(c) *Foundation permeability*. The permeability of the foundation (compared to that of the embankment) has an important effect on the seepage flownet. No dam foundation is 'impermeable' and in the majority of cases, even a rock foundation will have a permeability greater than that of compacted earthfill, e.g. most rocks have a permeability of between 1 and 100 lugeons (10^{-7} to 10^{-5} m/sec) compared to compacted clay earthfill with a $k_V \approx 10^{-7}$ to 10^{-10} m/sec. Soils in a dam foundation (which will usually be affected by fissures, root holes or layering during deposition) will almost invariably have a permeability far greater than compacted earthfill e.g. in the range 10^{-5} to 10^{-6} m/sec. Figure 11.14 shows the effect of such permeability contrasts on the seepage flownet for a earthfill embankment with a horizontal drain (Mardi Dam).



(a) Embankment dam with anisotropic permeability, $k_h = 15\ k_v$; equipotentials and pore pressures. (Fell, 1987)



(b) Embankment dam with isotropic permeability; equipotentials.

Figure 11.14 Effect of foundation permeability on seepage flow net at Mardi Dam (PWD of NSW, 1985) and Walker and Mohen, 1987).

The following further points are made on Mardi Dam:

i) Figure 11.14 shows the calculated seepage equipotentials for $k_H/k_V = 15$ in the embankment. These give pore pressures compatible with those observed in the embankment.

ii) For both cases (a) and (b) most seepage from the reservoir is through the foundation – equipotentials in the embankment upstream of the centreline are nearer horizontal than vertical.

iii) Even though k_H/k_V for the dam was ≈15 and seepage was emerging on the downstream face of the embankment, flow in the downstream part of the dam was towards the horizontal drain, yielding relatively low pore pressures. However the drain capacity was exceeded, so pore pressures built up in it.

The upper foundation layer has a lower permeability than the lower layer and the confining effect led to pore pressures in the lower layer being above ground surface at the downstream toe of the embankment. To remedy this situation pressure relief wells were constructed which fully penetrated the lower (sand) layer. The original wells shown in Figure 11.14 were only partially penetrating and therefore not very effective in relieving pressures.

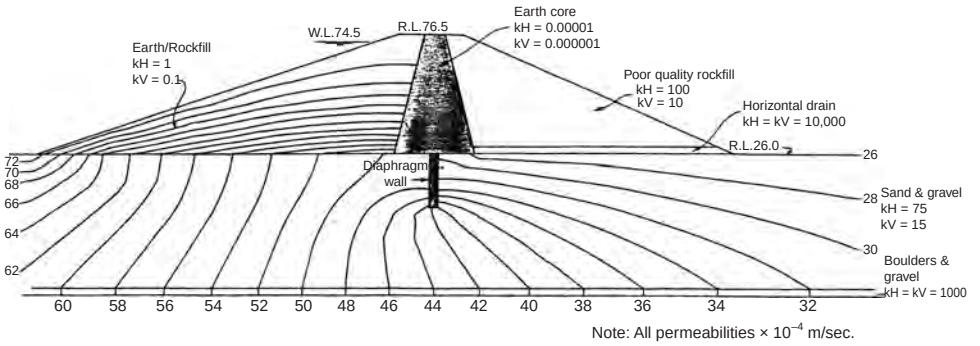
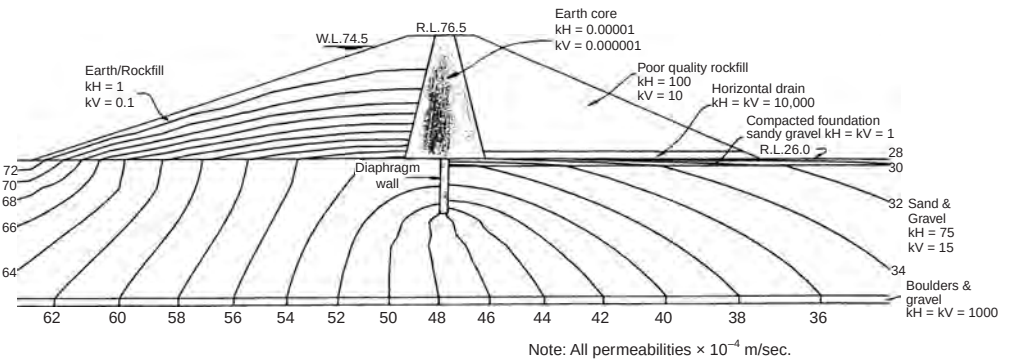
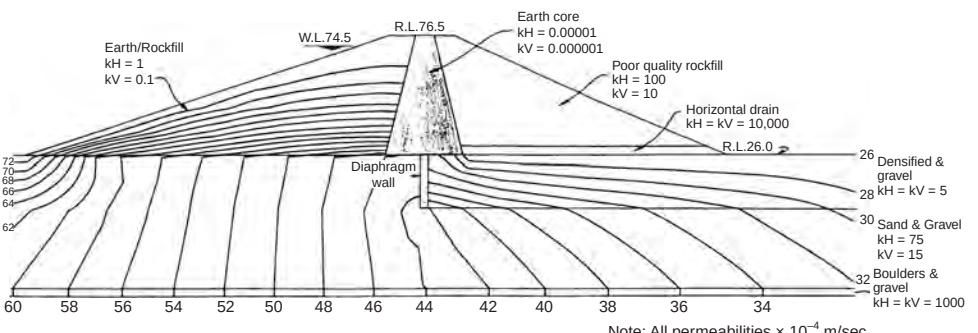


Figure 11.15 Seepage equipotentials Lungga Dam, uniform foundation (Coffey and Partners, 1981).



(a)



(b)

Figure 11.16 Seepage equipotentials Lungga Dam foundation affected by (a) upper compacted zone; (b) densification by vibroflotation (Coffey and Partners, 1981).

Figures 11.15 and 11.16 show the effect of foundation permeability on the seepage flow nets for Lungga Dam, Solomon Islands, which was to be constructed on permeable alluvial soils.

The analysis for Lungga Dam includes the use of a diaphragm wall which penetrates part way through the highly permeable foundation sands and gravels. Being partially penetrating it reduces seepage only by a small amount (less than 5%) compared to no diaphragm cutoff. Seepage is largely through the foundation (>99% of total) and the seepage through the upstream part of the dam embankment passes directly to the foundation. Figure 11.16(a) shows the flownet when the upper 3 m of alluvium was compacted, resulting in a lower permeability. Figure 11.16(b) shows the effect of vibroflotation of the foundation which by densification and mixing yields a lower permeability. Lungga Dam was not constructed, due to the high overall cost of the project.

It should be noted that for both Mardi and Lungga Dams, the critical failure surface for slope stability passed through the foundation due to the high pore pressures.

(d) *Dependence of permeability on void ratio and effective stress*. The permeability of a soil is dependent on the void ratio (e), and hence the mean effective stress (p′). This is discussed at some length in Vaughan (1994) who shows that the effect of including this phenomenon in seepage analyses is to give higher seepage gradients at the downstream of the core as shown in Figure 11.17. It is apparent that this should be modelled if correct predictions of pore pressures are required.

(e) *Effects of partial saturation*. Le Bihan and Leroueil (2000) develop the concept of St. Arnaud (1995) and demonstrate that the high gradients of pore pressure which occur near the downstream face of the core of some dams may be explained by the fact that the partially saturated permeability is lower than the saturated permeability. Hence as seepage begins to flow through the core of a dam, the flow regime is controlled by the contrasting permeability. It is hypothesised that the phenomenon is aided by dissolved gas in the seepage water coming out of solution in the lower furthermost downstream – part of the core where the pore pressures are lowest.

(f) *Clogging of the filters*. Peck (1990) suggests that fine particles from the core may migrate to the downstream filter, clog it and also lead to high pore pressure gradients on the downstream part of the core. This is more likely to occur if the soil in the core is non plastic and internally unstable (see Chapter 8).

From the above discussion it will be apparent that it is difficult to predict steady seepage pore pressures. There are several mechanisms which can lead to pressures equivalent to those where a high k_H/k_V ratio is assumed, i.e. reservoir head persists through the core. The authors' advice is to design all embankments on the assumption that k_H/k_V is ≥ 15. For larger, more critical dams, internal zoning should control seepage to such a degree that the stability is not greatly sensitive to k_H/k_V. The authors' own practice for an earth and rockfill embankment with a vertical drain is to ignore the flownet effect, i.e. assume the flow lines are horizontal in the core to the vertical drain. This is potentially conservative as it implies $k_H/k_V = \infty$ but only affects the factor of safety marginally over that if $k_H/k_V = 9$ were used and allows one to be confident that pore pressures have been estimated conservatively. In some cases this might allow adoption of a slightly lower factor of safety for the design or acceptance of a lower factor of safety for an existing dam, but some caution should be used in doing so.

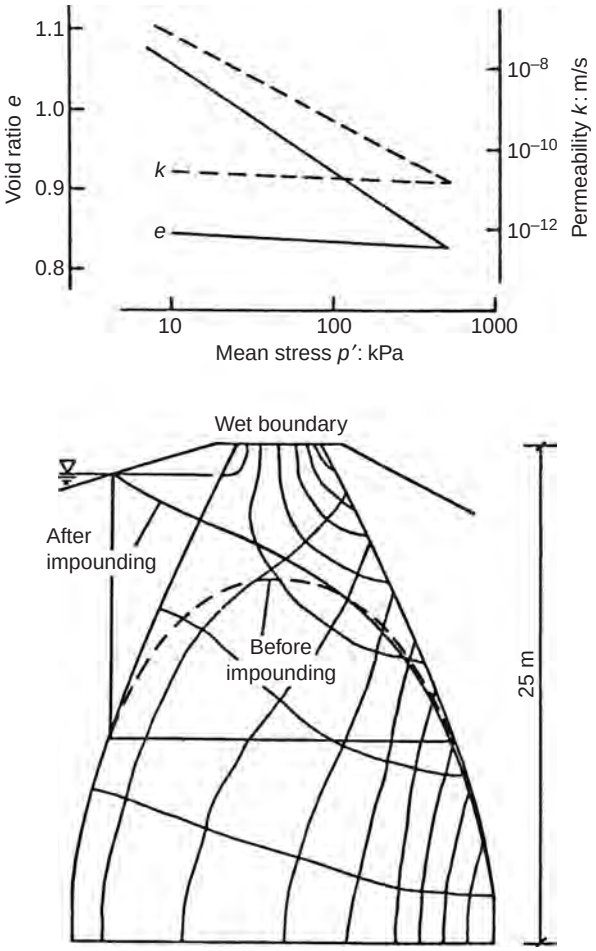


Figure 11.17 Seepage through a wide dam core predicted by coupled finite element analysis in two dimensions: $e \propto \log p'$ and $\ln k \propto e$ (Vaughan 1994).

For existing dams, there is often piezometer data on which to assess the pore pressures. The data need to be considered carefully, particularly in regard to whether equilibrium has been reached and whether there is sufficient instrumentation to define the pore pressures over the whole of the dam. The reliability of the existing pore pressure data should also be carefully considered. The implication of the high k_H/k_V on pressures should be considered.

11.4.1.2 *Pore pressures under flood conditions*

Pore pressures in the low permeability zones in a dam under flood conditions are very difficult to predict. There are two broad issues:

– Will pore pressures in the body of the dam, for example in the core, rise with the flood water level?
– Will pore pressures in the vicinity of the dam crest above normal Full Supply Level rise with the flood water level?

Considering these in turn:

(a) *Pore pressures in the body of the dam*. There are some general observations:
  i) Whether the pore pressures will respond rapidly to the reservoir rise depends on the degree of drainage which can be assessed as described in Section 11.4.3.3. If a zone is assessed to be free draining (Time Factor $T > 3$) or in the intermediate zone (Time Factor T 0.01 to 3), then it should be assumed the pore pressures will respond as the reservoir level rises in the flood.
  ii) For non-draining zones (time Factor $T < 0.01$) the pore pressures will respond slowly, so in most flood conditions there will be little or no rise in pore pressures.
  iii) Where monitoring of piezometers in the dam show a rapid response to reservoir level, regardless of the assessments above, assume the pore pressures will respond to the reservoir level.

(b) *Pore pressures near the crest of the dam*. The pore pressures near the crest of the dam will depend on whether the zones are free draining or not, as detailed above, but also on whether the core is affected by cracking due to differential settlement and desiccation. In many dams the low permeability zones have not been taken to dam crest end and high permeability zones, e.g. rockfill, are used, so these will saturate under the flood.

The authors are reluctant to assume that low permeability zones will not develop pore pressure during floods due to the possibility of cracking and would normally check the case where the pore pressures above Full Supply Level rise with the flood. If the dam is known to have cracking in the core, this would become the base design case. If there is no evidence or no knowledge of cracking, it is recommended it be assumed the pore pressures above FSL may rise with the flood, but that a slightly lower factor of safety might be considered acceptable for this condition. Caution should always be exercised in opting for a lower factor of safety, given the possible consequences of a massive failure of a dam with a reservoir in flood.

11.4.2 Pore pressures during construction and analysis of stability at the end of construction

11.4.2.1 *Some general principles*

When earthfill is compacted in a dam embankment the water content is usually controlled to be near standard optimum, so that compaction is facilitated. The soil will typically be compacted to a degree of saturation around 90% to 95% and will be heavily overconsolidated by rolling, with negative pore pressures in the partially saturated, compacted soil.

As additional fill is placed, the load imposed on the earthfill lower in the embankment will produce an increase in pore pressure, resulting eventually in positive pore pressures. The pore pressure will depend on the soil type, water content, roller used and magnitude of the applied stress (i.e. height of fill over the layer in question, and lateral restraint).

Larger dams constructed of earthfill with a high water content may develop sufficient pore pressure for this to control the stability of the upstream or downstream slope. If there is early closure of the diversion facility, the gradual rise in the storage may worsen the situation in so far as the upstream shoulder is concerned.

As the pore pressures will build up during construction and then (slowly) dissipate with time the critical condition may be during construction.

Another situation where construction pore pressures are important is where a dam is raised or when a berm is built to improve the stability. In these cases the core of the existing dam is usually saturated, so the pore pressure response will be that of a saturated soil and it is usual to analyse the stability using undrained strengths, which should be those existing in the dam, allowing for any swelling which may have occurred. This is discussed in Vaughan (1994).

The approaches which are available to consider the stability during construction are:

(a) Predict the pore pressures which will be developed using simplified methods such as Skempton (1954) and use effective stress stability analysis.
(b) Predict the pore pressures using more sophisticated partial saturated soil theory and use effective stress stability analysis or numerical analysis.
(c) Develop undrained strengths for the dam core and use total stress analysis.
(d) The following sections describe the Skempton (1954) approach, a variant on it by Vaughan reported in McKenna (1984), the status of partial saturated soil theory and the undrained strength approach.

11.4.2.2 *Estimation of construction pore pressures by Skempton (1954) method*

Construction pore pressures can be estimated by using Skempton's (1954) pore pressure parameters A and B. In this approach:

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{11.6}$$

where Δu = change in pore pressure; $\Delta\sigma_1$ = change in major principal; stress; $\Delta\sigma_3$ = change in minor principal stress.

A and B are pore pressure parameters, which are related to the compressibility of the soil as follows:

$$B = \frac{1}{1 + \dfrac{nC_v}{C_{sk}}} \tag{11.7}$$

where n = porosity; C_v = compressibility of the voids; C_{sk} = compressibility of the soil skeleton.

Table 11.5 Skempton 'A_f' values for clays of varying degrees of overconsolidation (Skempton, 1954).

Type of clay	A_f
Highly senstive clays	+0.75 to +1.5
Normally consolidated clays	+0.50 to +1.0
Compacted sandy clays	+0.25 to +0.75
Lightly overconsolidated clays	0 to +0.50
Compacted clay gravels	−0.25 to +0.25
Heavily overconsolidated clays	−0.50 to 0

The pore pressure equation can be rewritten as:

$$\frac{\Delta u}{\Delta \sigma_1} = \bar{B} = B\left[\frac{\Delta \sigma_3}{\Delta \sigma_1} + A\left(1 - \frac{\Delta \sigma_3}{\Delta \sigma_1}\right)\right] \tag{11.8}$$

For saturated soils, B = 1 but, as shown in Figure 6.14, B varies considerably with degree of saturation and the degree of over-consolidation of the soil. 'A' also varies with the degree of over-consolidation of the soil, the stress path followed and with the strain in the soil. Usually A at 'failure' (i.e. at maximum principal stress ratio or maximum deviator stress) is used and is known as A_f. Some typical values are shown in Table 11.5.

The values of A will also be dependent on the lateral strain conditions within the dam embankment. It is normal to assume that K_o conditions apply, i.e. that there is zero lateral strain. This is a reasonable (and conservative) assumption for the bulk of the failure surface in an embankment. The use of A at failure is also slightly conservative in most cases.

A more general pore pressure equation was proposed by Henkel (1960) to take into account the effect of the intermediate principal stress. It is:

$$\Delta u = B(\Delta \sigma_{oct} + a\Delta \tau_{oct}) \tag{11.9}$$

where

$$\sigma_{oct} = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) \tag{11.10}$$

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \tag{11.11}$$

and a is the Henkel pore pressure parameter. The equivalent Skempton A from Henkel's a parameter for triaxial compression is:

$$A = \frac{1}{3} + a\frac{\sqrt{2}}{3} \tag{11.12}$$

For triaxial extension conditions:

$$A = \frac{2}{3} + a\frac{\sqrt{2}}{3} \tag{11.13}$$

The pore pressure parameters A and B can be estimated by laboratory tests on the soil to be used in the embankment. 'B' is determined by placing soil compacted to the required water content and density ratio in a triaxial cell and observing the change in pore pressure Δu for changes in cell pressure under undrained conditions. Under these conditions $\Delta\sigma_1 = \Delta\sigma_3$ and the pore pressure equation becomes:

$$\Delta u = B[\Delta\sigma_3 + A(0)] = B\Delta\sigma_3 \tag{11.14}$$

This relationship will not be linear and must be determined over a range of cell pressures. B will be larger for higher cell pressures than for low cell pressures.

To determine A, the soil is placed in the triaxial cell under undrained conditions and sheared. Knowing B from the earlier testing and observing Δu, $\Delta\sigma_1$ and $\Delta\sigma_3$, A can be determined from the pore pressure equation. As explained above, in most cases this will be done under K_o conditions. Head (1985) shows a suitable test set up and lateral strain indicators needed to control the test using conventional triaxial equipment.

More accurate estimation of the pore pressure parameters can be achieved by more closely following the stress history of the soil in the embankment using a Bishop-Wesley triaxial cell.

To apply this method one must determine:

- What the stresses in the embankment will be.
- The pore pressure parameters A and B relevant to the water content and density ratio at which the soil is placed and the stress conditions in the embankment.

To be conservative in the estimation of pore pressures, the laboratory tests to estimate A and B can be carried out at the upper limit of specified water content (e.g. optimum +2%) and/or degree of compaction (density ratio 98% to 100%, standard compaction). However it must be remembered that at optimum +2% water content the maximum achievable density ratio may be 97% to 99%, while at optimum water content a density ratio of 100% to 101% may be achievable and the laboratory testing should account for this (see Figure 11.18).

The stresses in the embankment due to construction of the dam can be estimated by finite element techniques. ICOLD (1986a) and Duncan (1992, 1996a) give descriptions of the methods and their limitations. It is concluded that:

- A two dimensional model can be used to estimate vertical stresses in a homogeneous dam on a rigid (rock) foundation.
- A three dimensional model is necessary to model cross valley stresses in an embankment on a compressible foundation.
- The initial stresses locked into the fill during compaction should be incorporated into the model.
- The models must be 'built' in layers, being loaded progressively.

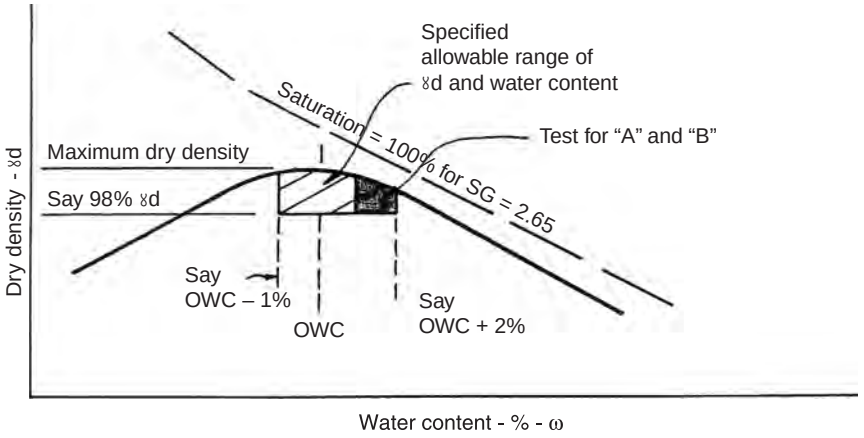


Figure 11.18 Test water content and density ratio for determining A and B parameters.

In a practical sense it is difficult to carry out 3 dimensional modelling and the rolling stresses can only be crudely modelled (Naylor, 1975 and ICOLD, 1986a). Given the difficulty in predicting pore pressure parameters A and B with any degree of accuracy, it is not worthwhile spending too much effort on modelling the stresses. Two dimensional finite element models along the dam axis, as well as up and down river, might give some appreciation of three dimensional problems for narrow or irregular shaped valleys and compressible foundations.

11.4.2.3 *Estimation of construction pore pressures from drained and specified undrained strengths*

McKenna (1984) cites an approximate method for estimating construction pore pressures developed by P.R. Vaughan. This method was used at Yonki Dam in Papua New Guinea by the Snowy Mountains Engineering Corporation and is applicable when the control of the embankment soil construction is by undrained shear strength, rather than by specifying a minimum dry density ratio and water content relative to optimum. This approach is used where the field water contents are high relative to optimum and drying of the soil is impracticable, or the optimum water content and maximum dry density are difficult to define.

The method is based on the effective strength and total strength envelopes obtained from tests on the embankment soil. These are plotted as p′-q plots, yielding envelopes where for the effective strengths:

$$q = p' \tan \alpha' + a' \tag{11.15}$$

and for total strengths:

$$q = p \tan \alpha + a \tag{11.16}$$

where $\tan \alpha' = \sin \phi'$; $a' = c' \cos \phi'$, $\tan \alpha = \sin \phi_u$; $a = c_u \cos \phi_u$.

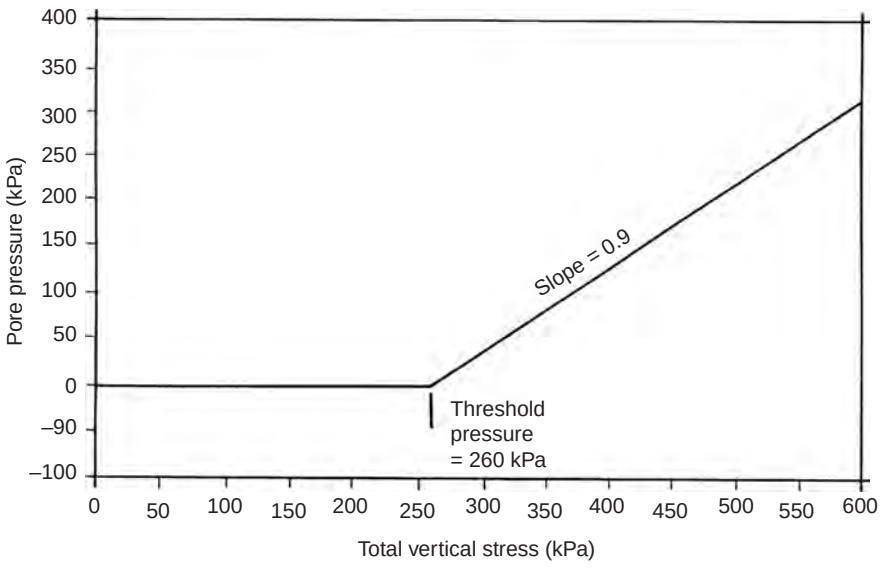


Figure 11.19 Predicted construction pore pressures vs total vertical stress for Yonki Dam using Vaughan method.

The difference between the total and effective stress envelopes represents the reduction in shear strength resulting from pore pressures developed on shearing.

The pore pressure at failure is given by:

$$u_f = \frac{a(1+\cot\alpha') - a'\cot\alpha'(1+\tan\alpha')}{(1-\cot\alpha'\tan\alpha')} + \frac{1-\cot\alpha'\tan\alpha'}{1+\tan\alpha'}\sigma_1$$

$$= \text{threshold pressure} + (\text{slope of curve})\sigma_1 \qquad (11.17)$$

In this σ_1 is the total vertical stress in excess of the threshold pressure, e.g. for $c' = 0$, $\phi' = 30°$ and $\phi_u = 2°$; slope of curve = 0.9, 'x' intercept = 3.23Su.

Hence if the dam is constructed so that Su = 80 kPa, the predicted pore pressures would be as shown in Figure 11.19.

In practice there are negative pore pressures in the embankment below the threshold pressure and the slope of the curve increases with increasing vertical stress. Figure 11.20 shows results from Yonki Dam.

11.4.2.4 *Estimation of pore pressures using advanced theory of partially saturated soil*

Fry *et al.* (1996) present a good overview of the state of art of application of partially saturated soil mechanics to embankment dams and slope stability up to 1996.

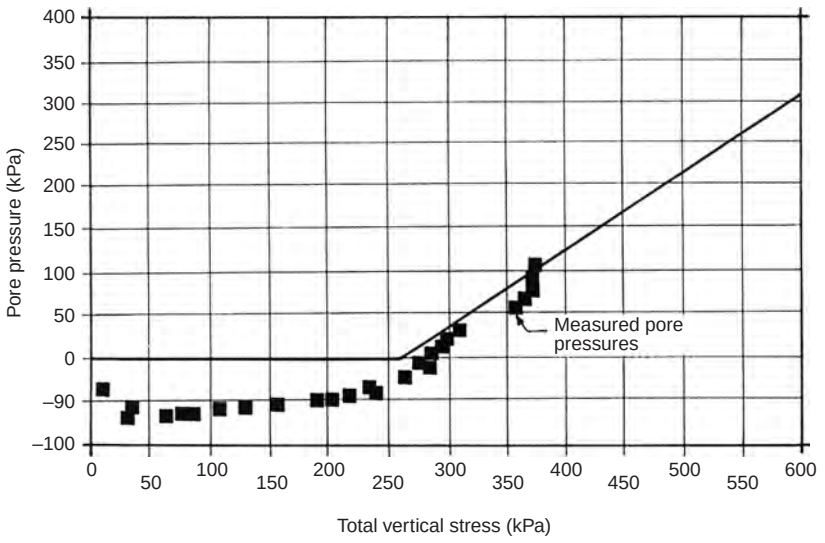


Figure 11.20 Observed construction pore pressures for Yonki Dam – courtesy of the Snowy Mountains Engineering Corporation.

As they point out, there are two main approaches:

(a) Uncoupled analysis, where the first step is to model the pore pressures using partially saturated flow equations, followed by the use of limit equilibrium analysis. These analyses allow, with varying degrees of sophistication, for the dependence of the unsaturated permeability with the water content, stresses, degree of saturation and diffusivity (equivalent to the coefficient of consolidation in saturated soils). The many shortcomings of such analyses are detailed by Fry *et al.* (1996).

(b) Coupled analysis models (again with varying degrees of sophistication) the pore air and water pressures, volume change and stresses using finite element analyses and properties based on what are usually time consuming and sophisticated laboratory tests. Fry *et al.* (1996) provide a review of the approaches available at that time.

Now (2014) there is considerable research in progress on this topic which should provide better methods. In view of this and the reality that the science of partially saturated soils is well beyond most dam engineers (and the authors!), if it is wished to attempt a fully coupled approach, one of the handful of experts in this field should be engaged to assist.

11.4.2.5 *Undrained strength analysis*

In view of the difficulty in correctly estimating the pore pressures in partially saturated soils, for most dams it is sufficient to use the undrained strengths (for cohesive soils) and a total stress analysis.

The undrained strengths used are usually those estimated to apply at the placement condition, so any strength gained by (partially saturated) consolidation as the dam is constructed are ignored.

Alternatively, the undrained strengths can be determined by a series of laboratory tests which consolidate the samples to the effective stresses expected in the dam. These stresses should be estimated by numerical analysis and the testing should follow the stress path of the soil from compaction through to the conditions in the dam.

Just as there are uncertainties in estimating the pore pressures, there are uncertainties in these undrained strengths.

For raising existing dams, carry out CPTU and laboratory triaxial testing to evaluate the undrained shear strengths of the existing fill materials.

11.4.2.6 *Summing up*

It should be recognised that it is not practicable to estimate construction pore pressures accurately and that the best one can do is to get an indication of the likely pressures, and how they will build up as the embankment is constructed. It is necessary to monitor pore pressures with piezometers if it is considered they may be critical. Measures can be taken to reduce pore pressures, e.g. by using a lower water content. This was done at Dartmouth Dam (Maver *et al.*, 1978).

It is also important to monitor displacements of the dam during construction, so the onset of increasing rates of movement, possibly indicating construction instability, can be detected. Surface survey should always be possible. Bore hole inclinometers are valuable but some, including the authors, are reluctant to put too much instrumentation into dam cores because of the increased chances of piping problems along the instruments.

The discussion above, with the exception of Section 11.4.2.4, ignores the effects of dissipation of pore pressure with time in the embankment. Eisenstein and Naylor in ICOLD (1986a) discuss how this can be modelled by finite element methods. In most cases the earthfill will have a low permeability and coefficient of consolidation and drainage paths will be long which will result in long times for pore pressure dissipation. As the estimation of pore pressures is approximate, it would not be warranted to take account of pore pressure dissipation in most cases.

In most cases, it will be sufficient to use the undrained strengths (for cohesive soils) and a total stress analysis rather than relying on predicting construction pore pressures for an effective stress analysis.

11.4.3 Drawdown pore pressures and the analysis of stability under drawdown conditions

11.4.3.1 *Some general issues*

When the reservoir level behind an embankment dam is lowered, the stabilizing influence of the water pressure on the upstream slope is lost. If the water level is dropped sufficiently quickly that the pore pressures in the slope do not have time to reach equilibrium with the new reservoir water level, the slope is less stable.

Many failures of dam slopes (and natural slopes around reservoirs) have occurred during drawdown. Duncan *et al.* (1990) give some examples.

It is often assumed, for design purposes, that the drawdown is rapid or even instantaneous. This assumption imposes severe loadings and it is often the controlling case for the design of the upstream slope.

Terzaghi and Peck (1967) summarized the problems in estimating pore pressures during drawdown in these words:

> *"... in order to determine the pore pressure conditions for the drawdown state, all the following factors need to be known: the location of the boundaries between materials with significantly different properties; the permeability and consolidation characteristics of each of these materials; and the anticipated maximum rate of drawdown. In addition, the pore pressures induced by the changes in the shearing stresses themselves ... need to be taken into consideration. In engineering practice, few of these factors can be reliably determined. The gaps in the available information must be filled by the most unfavorable assumptions compatible with the known facts."*

In view of the difficulty in estimating the pore pressures, particularly those developed on shearing, in low permeability, clayey soils it is common practice to use an essentially total stress analysis using undrained strengths to assess the stability under drawdown. The recommended procedure is detailed in Section 11.4.3.3. For free draining rockfill or gravels, the pore pressures will dissipate as the reservoir level drops, so there are no residual pore pressures.

For well compacted, medium permeability soils, e.g. silty sands, pore pressures can be estimated using flow nets. In these cases any shearing induced pore pressures will be negative, due to dilation of the dense soil, so they can be conservatively ignored. If the silty sands are loose and drawdown is in fact rapid, undrained strengths should be used.

Duncan (1992) gives guidance on how to assess whether drainage will occur. This is covered in Section 11.4.3.3.

11.4.3.2 *Estimation of drawdown pore pressures, excluding the effects of shear-induced pore pressures*

Pore pressures in the upstream slope of a dam under drawdown conditions are determined by the change in water level, geometry and zoning of the slope, the relative permeabilities of the zones and, importantly, layers within each zone and the relative permeability of the foundation.

Some of these points are illustrated in Figure 11.21 and discussed below:

i) In most cases the assumption is made that drawdown is 'instantaneous', and hence the flow net changes from the steady state case to the drawdown case instantaneously. This is somewhat conservative but sufficient for most analysis.
ii) The pore pressures in the embankment are immediately reduced on drawdown due to the unloading effect of removing the water. Hence pore pressures on the surface of earthfill in Figures 11.21a to 11.2d are all zero.
iii) In Figure 11.20a the foundation permeability is low compared to that of the earthfill and flow will be towards the upstream face. This gives relatively high pore pressures compared to Figure 11.21b. In the latter case, the foundation

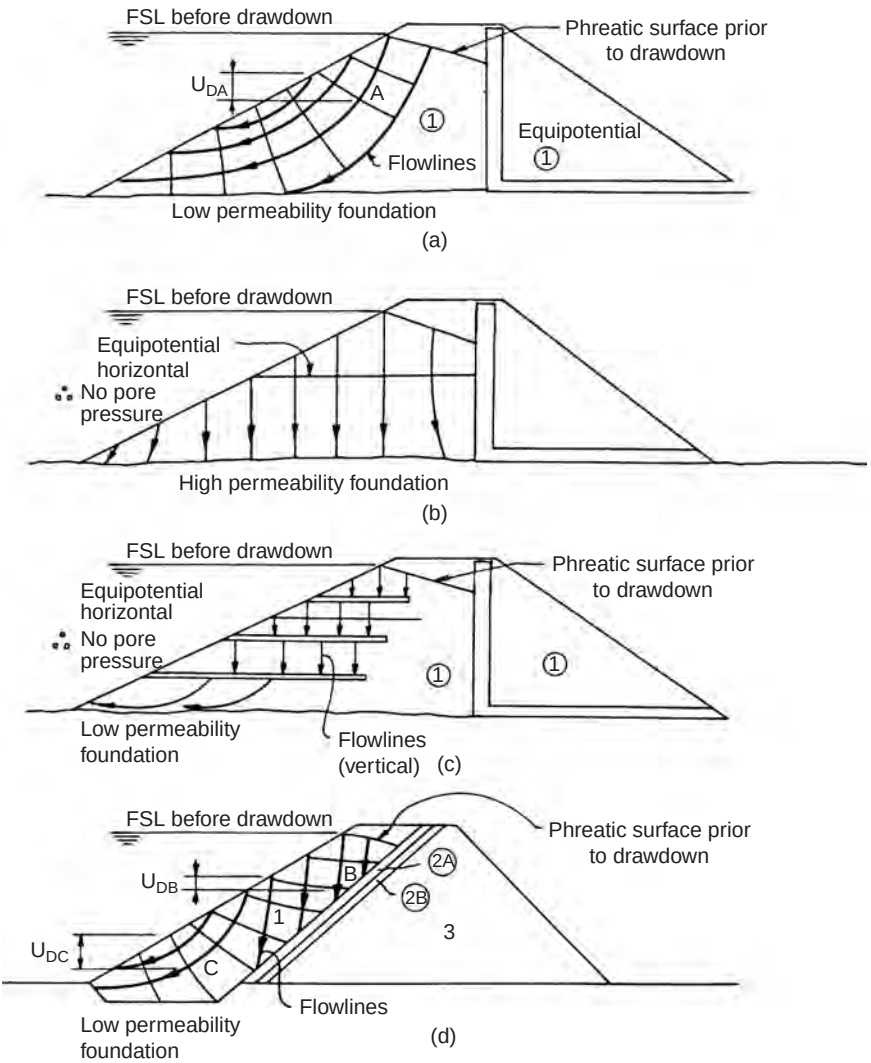


Figure 11.21 Drawdown flow nets (a) earthfill with chimney drain, low permeability foundation; (b) earthfill with chimney drain, high permeability foundation; (c) earthfill with horizontal drainage blankets; (d) sloping upstream core, low permeability foundation. Note that flow nets assume isotropic permeability and $k_H = k_V$ and the reservoir is completely drawn down.

is more permeable than the earthfill, so flow lines will be near vertical and equipotentials near horizontal, giving pore pressures which should be close to zero.

iv) In Figure 11.21c horizontal drains are incorporated in the upstream slope. These cause flow lines to be near vertical on drawdown (except near the low permeability foundation) and should result in low pore pressures.

v) In Figure 11.21d the flow lines are influenced by the presence of the filter zones and rockfill, giving lower pore pressures than for Figure 11.21a, at least in the upper part of the slope.

The diagrams in Figure 11.20 are sketched with the implicit assumption that $k_H/k_V = 1$ and permeability is isotropic (uniform). In reality, if there are lower permeability layers within the zones, perched water tables may result, completely altering the pore pressures. Hence great care must be taken in the analyses and it would be more common to assume that the pore pressures are those which exist prior to the drawdown, adjusted for the unloading effect of removing the water.

Drawdown flow nets and pore pressures can be obtained by graphical methods or finite element analysis. Factors which must be considered in the analyses are:

- Permeability anisotropy in the earthfill, i.e. allowing for $k_H > k_V$ and low permeability layers.
- The relative permeability of the foundation and the embankment zones. In practice this means that the foundation should be modelled with the embankment.
- The effect of removal of buoyancy from the upstream rockfill zone (if present) should be allowed for. This will induce increased pore pressures.

The drawdown condition is the controlling case for design of the upstream slope of embankments such as those shown in Figure 11.21 and the analysis is sensitive to the assumptions made. For a central core earth and rockfill dam the analysis may control the slope but will not be so sensitive to assumptions.

11.4.3.3 *Methods for assessment of the stability under drawdown conditions*

Duncan *et al.* (1990) discuss several methods for analysing the stability of the upstream slope under drawdown conditions, including those developed by Bishop (1955), Morgensten (1963), US Corps of Engineers (1970) and Lowe and Karafiath (1960). These latter two methods have in common that they use a composite drained and undrained strength envelope. Duncan *et al.* (1990) favour these methods in preference to effective stress methods which rely on estimating the pore pressures generated by shearing. As discussed above this is very difficult.

They recommend a procedure which is a development of the method developed by Lowe and Karafiath (1960) and is somewhat complicated in that it is iterative and requires an estimate of the principal stress ratio at the base of each slice to assist in selecting the undrained strength. While it seems quite reasonable and logical, the complexity of its application makes it difficult to recommend its use. In view of this, it is recommended that the procedure given below, which is based on USBR (1987) and Duncan *et al.* (1990) be used.

The suggested approach is:

(a) Assess for each zone in the dam whether drainage will occur during drawdown, using the approach suggested by Duncan (1992). Estimate the dimensionless time factor T from:

$$T = \frac{C_v t}{D^2} \tag{11.18}$$

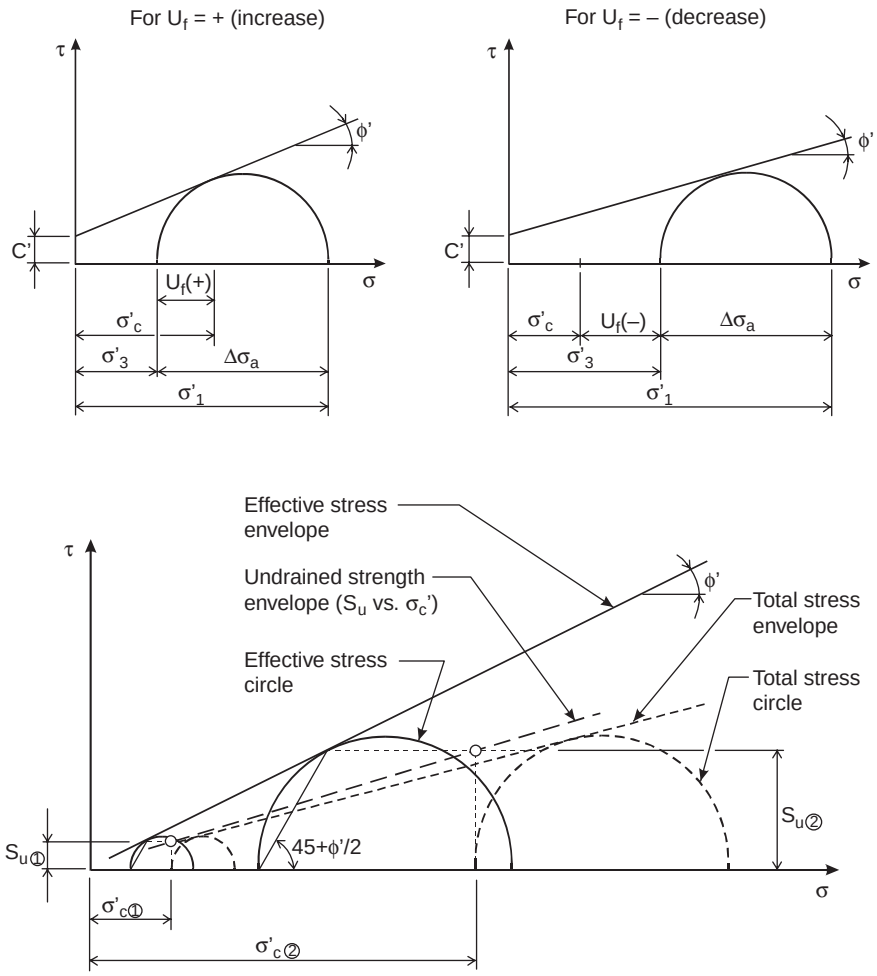


Figure 11.22 Shear strength envelopes for isotropically consolidated undrained (IC-U) strength tests.

where C_v = coefficient of consolidation (m^2/year); t = time for drawdown (years); D = drainage path length (m).

If T > 3.0, the zone will be free draining, so effective stress parameters will apply.

If T < 0.01, the zone will be undrained, so undrained strengths should be used.

If >0.01 and <3, the zone will be partially drained. For drawdown analysis assume the zone will be undrained.

(b) For the zones which are below normal maximum reservoir operating level, e.g. below Full Supply Level, develop effective stress and total stress strength envelope from conventional isotropically consolidated undrained (IC-U)triaxial tests, as shown in Figure 11.22.

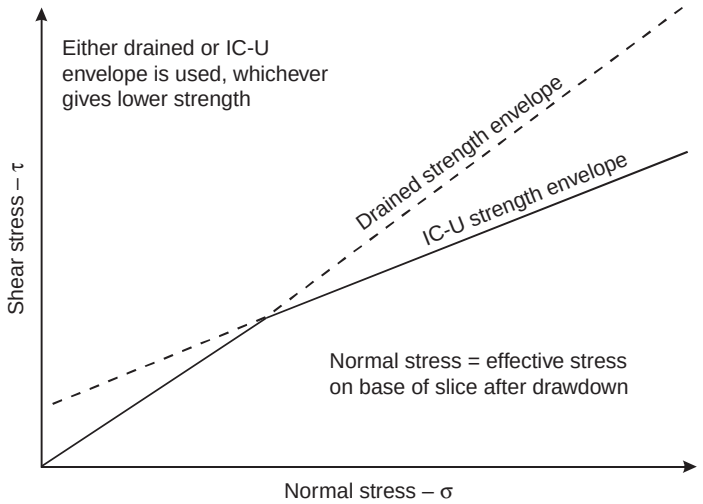

Either drained or IC-U
envelope is used, whichever
gives lower strength
Drained strength envelope
IC-U strength envelope
Normal stress = effective stress
on base of slice after drawdown
Shear stress – τ
Normal stress – σ

Figure 11.23 Composite shear strength envelope.

Note that the effective stress Mohr circle will be to the left or right of the total stress circle, depending on whether the pore pressures generated during shear are positive or negative.

(c) For these zones, develop the composite shear strength diagram shown in Figure 11.23, from the effective stress and undrained strength envelopes in Figure 11.22.

(d) For the zones which are above normal maximum operating level, e.g. above FSL, estimate the undrained strengths from the undrained strength envelope, allowing for any known cracking or softening.

(e) For undrained or partially drained zones below the normal maximum operating level, determine the strength from the composite envelope, i.e. the minimum of effective stress and undrained strengths. The normal stress used to select the shear strength from the composite envelope should be the effective consolidation stress acting on the base of the slice after drawdown, i.e. the total normal stress minus the pore pressure taken from the drawdown flow net (or from measured pore pressures), after allowing for the factors discussed in Section 11.4.3.2.

This procedure has the following characteristics:

i) It uses undrained strengths, where these are less than effective stress strengths, to allow for the effects of pore pressures developed during the undrained loading.

ii) It uses effective stress strengths, where these are lower than undrained strengths, in areas normally submerged by the reservoir. This is as recommended by USBR (1987), US Corps of Engineers (1970), and Duncan *et al.* (1990), and is to allow for possible drainage (the drainage paths may be short in these low stressed areas) and softening under the low confining stresses. It will also allow for the cracking, dilation and softening of the earthfill which occurs because of settlements during construction and under repeated drawdown events.

iii) It uses the effective stress after drawdown to estimate the strengths as recommended by Duncan *et al.* (1990). They demonstrated that the use of the effective stresses prior to drawdown as recommended by US Corps of Engineers (1970) (and USBR, 1987) is conservative. However the authors caution that these effective stresses should be estimated conservatively and the default position would be to use the effective stress prior to drawdown which is more readily estimated.

iv) It uses undrained strengths above the normal maximum operating level (after allowing for cracking and softening) because effective stress strengths in this area, where confining stresses are low, greatly under-estimate the real strength of the partially saturated soil and, if used, could give misleadingly low strengths in the upper part of the dam. Dams which have experienced larger than normal settlements should be assumed to have cracking and softening present.

11.4.3.4 *Some detailed issues for drawdown analyses*

(a) When analysing the stability of the upstream slope of a dam it is important to consider shallow and deep failure surfaces. Shallow surfaces are more likely to drain during the reservoir drawdown so drained parameters may be most appropriate. Deeper surfaces are less likely to drain so undrained strengths may apply. In most cases shallow surfaces will have lower factors of safety but depending on the embankment geometry and freeboard may be unlikely to breach the dam. Deeper surfaces may be more likely to result in loss of freeboard at the crest, although in most cases the potential for this to lead overtopping and breach will be very low as the sliding occurs when the reservoir level is drawn down. However, consideration needs to be given to the potential for the surfaces to causing cracking of the core and displacement of filter zones and the potential consequential development of the piping and internal erosion. Note that while upstream slope failure surfaces may not result in a breach of the dam they could still mean that the dam and reservoir must be taken out of service for a considerable time so that repair works can be done. Such an outcome should be taken into account when deciding if shallow failures could be acceptable.

(b) Multiple reservoir drawdowns often result in increased deformation of the upstream slope. This deformation is likely to result in some strain softening giving in effective stress terms weakening from peak to fully softened strength as a minimum, and potentially towards residual strength. It is also likely to result in longitudinal cracking in the crest of the dam, and this cracking should be modelled in the stability analyses. It should be assumed that the cracks will fill with water under rain events following the drawdown. The slope failure at San Luis Dam in California is such a case.

11.5 DESIGN ACCEPTANCE CRITERIA

11.5.1 Acceptable factors of safety

Most dams are designed and the safety of existing dams is assessed using factors of safety as acceptance criteria. The factors of safety adopted are reasonably universal,

Table 11.6 Baseline recommended minimum acceptable factors of safety and load conditions.

Slope	*Load Condition*	*Reservoir Characteristic*	*Minimum Factor of Safety*
Upstream and Downstream	End of Construction Steady state seepage	Reservoir empty	1.3
Downstream	Maximum flood	Reservoir at normal maximum operating level (Full Supply Level)	1.5
Downstream	Drawdown	Reservoir at maximum flood level	1.5, free draining crest zones, 1.3 otherwise
Upstream		Rapid drawdown to critical level	1.3

Notes: (1) These factors of safety apply to design of new high consequence of failure dams, on high strength foundations, with low permeability zones constructed of soil which is not strain weakening, using reasonably conservative shear strengths and pore pressures developed from extensive geotechnical investigations of borrow areas, laboratory testing and analysis of the results and using the methods of analysis detailed above. It is assumed there will be monitoring of deformations by surface settlement points during construction and during operation of the dams.
(2)"High permeability crest zones" means the pore pressures in zones near the crest will respond to reservoir level as it rises. For dams with a low permeability earthfill core the pore pressures will not respond to reservoir rise and lower factors of safety may be acceptable.

with for example similar values used by US Bureau of Reclamation (USBR, 1987), 2013, US Corps of Engineers (2003), Building Research Establishment Guide to the Safety of Embankment Dams in the United Kingdom (BRE, 1990) and Norwegian Geotechnical Institute (1992).

Table 11.6 lists what can be regarded as baseline minimum factors of safety applicable to design of new dams with high consequence of failure, with the qualifications listed in the notes.

Table 11.7 details factors which should influence the selection of a minimum acceptable factor of safety and their extent of influence.

Tables 11.6 and 11.7 are based on the following:

(a) The strengths and pore pressures used in the analysis should be selected reasonably conservatively, e.g. use lower quartile shear strengths; high k_H/k_V in compacted earthfill. This is always a matter of judgement and the authors' recent experience is that practitioners are sometimes overly conservative in selection of strengths, e.g. using lower bound strengths for earthfill, not allowing for the high friction angles of rockfill at low confining stresses.
(b) The lower minimum factors of safety for the construction condition is based on the consequences of failure usually being lower than for a dam under steady state seepage. However the uncertainties in the shear strength are often high, so unless there is good monitoring of pore pressures and displacements, high factors of safety should be used.
(c) The lower factors of safety for rapid drawdown are again predicated on the assumption that the dam is unlikely to breach under a drawdown failure. It should be recognised that by adopting lower factors of safety for the construction and drawdown conditions there is an acceptance of a higher probability of instability.

Table 11.7 Factors which influence the selection of factor of safety and their effect on the baseline minimum factor of safety.

Factor	*Description*	*Recommended Change to the baseline Minimum Factor of Safety*
Existing (vs new) Dam	A lower factor of safety may be adopted for an existing dam which is well monitored and performing well	0 to −0.1
Soil or weak rock foundation	A higher factor of safety may be needed to account for the greater uncertainty of the strength	0 to +0.2 for effective stress +0.1 to +0.3 for undrained strength analyses.
Strain weakening soils in the embankment or foundation	A higher factor of safety may be needed to account for progressive failure, and greater displacements if failure occurs	0 to +0.2
Limited (little or no good quality) strength investigation and testing, particularly of soil and weak rock foundations	A higher factor of safety should be used to account for the lack of knowledge. Detailed investigations will be required to confirm conditions.	+0.1 to +0.3 for effective stress analyses, +0.3 to +0.5 for undrained strength analyses.
Contractive soils in the embankment or foundation	A higher factor of safety may be needed to account for the greater uncertainty in the undrained strength	+0.1 to +0.3 for undrained strength.

Note: These figures are given for general guidance only. Experienced Geotechnical Professionals should use their own judgment, but note the principles involved in this table.

(d) The factors of safety apply to non-trivial failure surfaces. Failure surfaces passing only through free draining rockfill may acceptably have lower factors of safety, because provided the slope can be constructed; there is no possible change in conditions which will cause instability. If realistic (high) effective friction angles are used for rockfill at low confining stresses, this problem will generally not arise.

(e) The greater uncertainty in undrained strengths than in effective stress strengths means that higher factors of safety should be used where undrained strengths control stability.

11.5.2 Post failure deformation assessment

It is recommended that as a matter of routine the potential post failure deformation of the dam be assessed using the simplified method of Khalili *et al.* (1996).

The Khalili *et al.* (1996) model considers two modes of failure defining the upper and lower bounds of post-failure deformation:

– A 'rapid model' based on the principle of conservation of energy, where the potential energy of the slide mass is resisted by the frictional forces along the surface of rupture, and approximated using the residual shear strength.

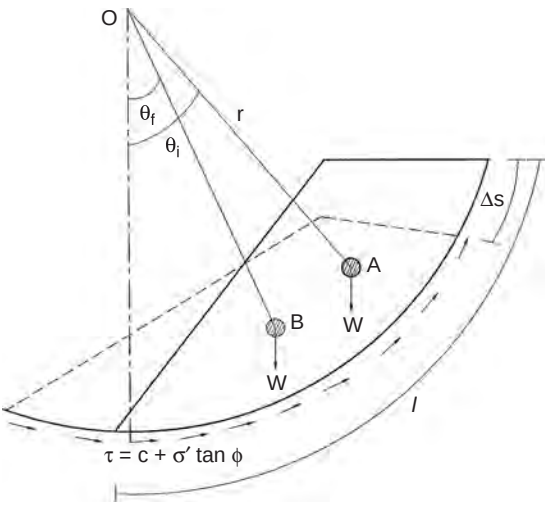


Figure 11.24 Failure model for post-failure deformation (Khalili *et al.*, 1996).

– A 'slow model' based on equations of equilibrium between the driving force of the slide mass and resisting forces along the surface of rupture and approximated using the residual shear strength, i.e. assuming the rate of movement would be so slow that the effect of inertia forces of the sliding mass is negligible.

The failure model is shown in Figure 11.24 and is based on the assumptions of a circular slide surface, the failure is plane strain, the failing mass moves as a rigid body and energy losses during failure is only due to the frictional forces acting along the slip surface. Approximate solutions for the rapid model are calculated using Equation 11.19 and for the slow model using Equation 11.20, where $FS_{residual}$ is the factor of safety calculated using residual strengths along the surface of rupture, and θ_i and θ_f are the initial and final positions of the centre of gravity as defined in Figure 11.24.

$$\text{Rapid Model } \theta_f = 2\theta_i(FS_{residual} - 0.5) \tag{11.19}$$

$$\text{Slow Model } \sin\theta_f = FS_{residual} \sin\theta_i \tag{11.20}$$

Khalili *et al.* (1996) found that for most cases analysed the rapid model gave the best estimate of deformations.

In this manner, the consequences of the dam 'failing' (reaching a factor of safety ≤ 1) can be determined. It will become apparent that in many cases, either the factor of safety using residual strengths is ≥ 1.0, in which case failure is virtually impossible, or it is only a little less than 1.0, because the materials in the dam are not significantly strain weakening.

If this is the case, and if the assessment of the post failure deformation is such that freeboard will not be lost, one might, after careful consideration, accept a lower factor of safety than usually adopted rather than carry out remedial works.

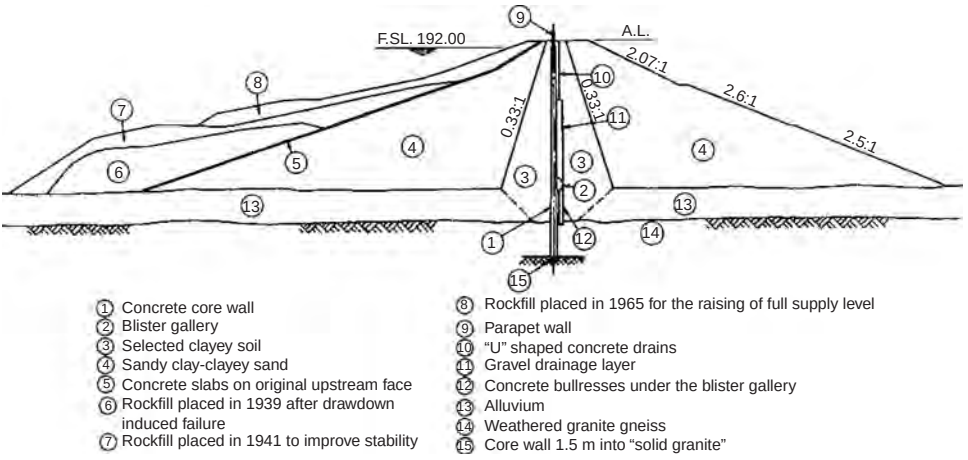

F.SL. 192.00
A.L.
2.07:1
2.6:1
2.5:1
0.33:1
0.33:1
1 Concrete core wall
2 Blister gallery
3 Selected clayey soil
4 Sandy clay-clayey sand
5 Concrete slabs on original upstream face
6 Rockfill placed in 1939 after drawdown induced failure
7 Rockfill placed in 1941 to improve stability
8 Rockfill placed in 1965 for the raising of full supply level
9 Parapet wall
10 "U" shaped concrete drains
11 Gravel drainage layer
12 Concrete bullresses under the blister gallery
13 Alluvium
14 Weathered granite gneiss
15 Core wall 1.5 m into "solid granite"

Figure 11.25 Hume No. 1 Embankment Cross Section (Cooper *et al.*, 1997).

Situations which are likely to be non-brittle (low chance of large deformations) include dams with large rockfill or dense sand-gravel zones and/or earthfill with low clay content (less than ≈20% based on Figure 6.5).

Situations which are likely to be brittle are those with little or no rockfill and brittle, strain weakening soils, as described in Section 6.1.3.

11.6 EXAMPLES OF UNUSUAL ISSUES IN ANALYSIS OF STABILITY

The two case studies detailed below exemplify why conventional effective stress analysis has shortcomings in some situations.

11.6.1 Hume No. 1 Embankment

Hume No. 1 Embankment is a 39 m high, 1170 m long earthfill dam, with a concrete core wall which is 0.6 m thick at the top and up to 1.8 m thick at the base. The core wall is lightly reinforced. Figure 11.25 shows the cross section and description of the zones.

The embankment was constructed in the period 1921–1936 and rockfill was placed on the upstream slope in 1939 and 1941 following upstream slope failures on reservoir drawdown. The dam was raised 5.5 m in the period 1953–1965. The earthfill in the original construction was placed by horse and dray with the horses' hooves providing the only compaction.

Monitoring of the dam (Mail *et al.*, 1996) from 1966 showed downstream movements of the top of the core wall totalling 450 mm in one area, with 20 mm in 1993 alone. The movements only occurred at high reservoir levels.

Detailed investigations and analyses were carried out to determine the reason for the deformations. These are described in Cooper *et al.* (1997)and included drilling

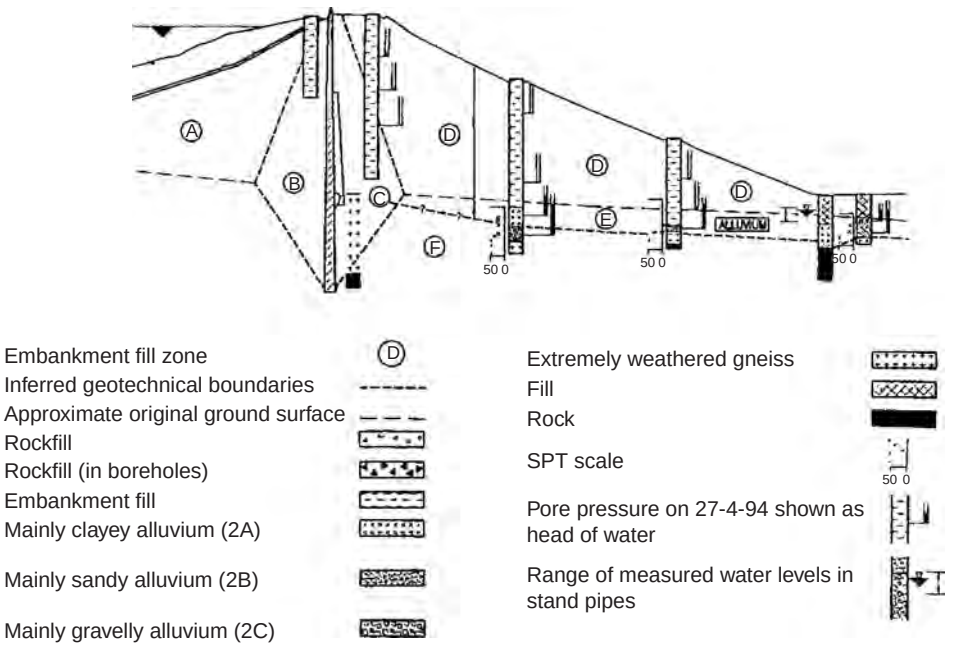


Figure 11.26 Hume No. 1 Embankment, Bend area – Geotechnical conditions (Cooper *et al.*, 1997).

boreholes, installing piezometers, cone penetration (CPT) and piezocone (CPTU) testing, laboratory testing and limit equilibrium and numerical modelling of stability.

Figure 11.26 shows the geotechnical conditions in the bend area which deformed excessively.

The earthfill downstream of the concrete core wall was generally unsaturated, with local perched water tables. The lower 2 m of the fill was saturated by seepage through the abutment or leakage through the core wall. The CPT showed the saturated zones had much lower undrained shear strength than the rest of the fill.

Limit equilibrium stability analyses were carried out using effective stress (peak and residual) and undrained strengths. The undrained strengths were estimated from the CPT, CPTU and, more particularly for the saturated zones, from the effective stress strengths taking account of pore pressures generated during undrained shearing using the formula:

$$Su = \frac{c' \cos \varphi + Po \sin \varphi}{1 + 2A_f \sin \varphi'} \tag{11.21}$$

$$\text{where } Po = \frac{1 + Ko}{2} \sigma'_{vo} \tag{11.22}$$

where c', ϕ' are the effective stress peak strengths, A_f = Skempton's pore pressure coefficient at failure, Ko = the at rest earth pressure coefficient, σ'_{vo} = effective vertical stress. A_f was in the range 0.36 to 0.7 for the triaxial tests. A value of 0.7 was adopted.

Table 11.8 Hume No. 1 Embankment Results of Limit Equilibrium Stability Analysis (Cooper *et al.*, 1997).

	Factors of Safety	
Case	*Effective Stress*	*Undrained Strength*
Peak strengths	1.52 (1.52)*	1.23 (1.14)
Residual strength	1.06 (0.98)	0.97 (0.89)

Note: The figures in brackets assume no strength in the concrete core. For values marked '*', the critical failure surface does not pass through the wall.

The results of the analysis are given in Table 11.8.

The numerical analysis underestimated the actual displacements using the undrained strengths derived from peak strengths and overestimated them using the undrained strengths derived from the residual strengths. The analysis showed a yield surface developed in the 2 m of saturated fill just above the foundation downstream of the core wall and was able to replicate movements at other parts of the dam not affected by the saturation of the lower part of the fill.

From this it was concluded that:

- The behaviour of the dam was due to marginal slope stability.
- The use of effective stress analysis significantly over-estimated the factor of safety.
- Undrained strength analysis more correctly modelled the behaviour but allowance must also be made in this case for strain weakening.

It should be noted that the investigations had shown that the movements could not be explained by consolidation, collapse mechanisms or internal erosion and piping.

Subsequent investigations of an area where No. 1 Embankment joins the concrete spillway section confirmed the need to use undrained strengths in stability analysis. In that area borehole inclinometers confirmed the development of shear surfaces in the saturated, softened fill. Both areas have had berms constructed to improve the stability.

11.6.2 Eppalock Dam

Eppalock Dam is a 47 m high central core earth and rockfill dam which was built between 1960 and 1962. Figure 11.11 shows a cross section through the dam.

The rockfill in the dam was placed in lifts 2 m to 4 m thick, spread with a light tractor. There was no rolling. The core was compacted by sheepsfoot roller, probably to >98% density ratio (standard compaction). A weighting berm was built on the upstream side from waste rock and soil to improve the stability relating to potential weak zones in the foundation. The berm material was probably not well compacted.

Since monitoring began in 1962, the dam experienced relatively large post construction deformations (vertically up to 850 mm, laterally 750 mm). Figure 11.11 shows typical movement vectors plotted on a time basis. It will be noted that the overall effect is one of spreading and that the displacement on the crest was downstream until about 1978, thence it has been upstream.

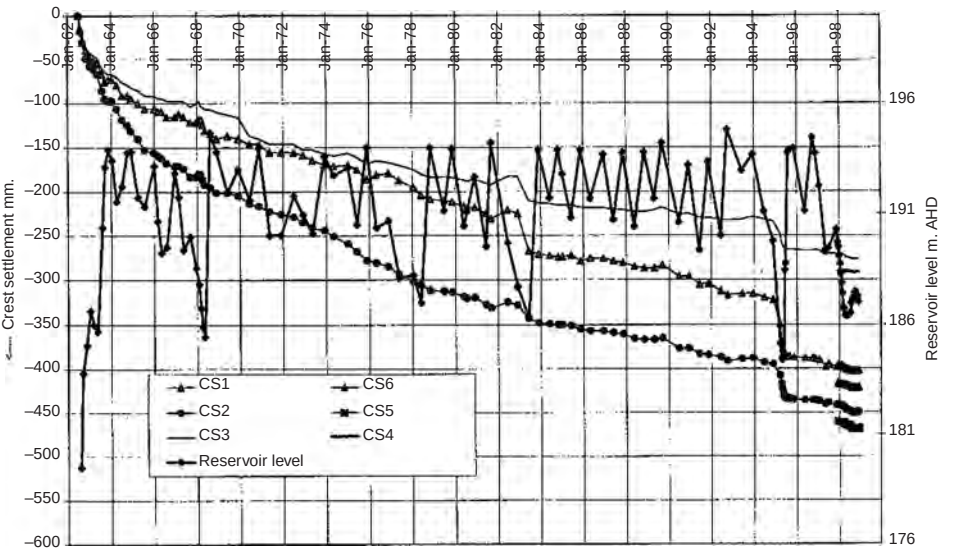


Figure 11.27 Eppalock Dam – Observed crest settlement (Courtesy of Goulburn Murray Water).

Figure 11.27 shows the observed settlements on the dam crest. The lateral movements were also measured. It will be noted that there is a general trend for on-going deformation which was reducing with time, but there have been rapid movements in 1968, 1983 and 1995. These relate to low reservoir levels and appear to coincide with rainfall events at the end of the reservoir drawdowns. The movements increased with each drawdown except that in 1998 the movement was less. This is probably because of a rigorous procedure of filling cracks in the dam crest as they formed, stopping rainfall infiltration.

In 1995 there was severe cracking and disruption of the dam crest and longitudinal cracks were observed in the other drawdown periods. Extensive investigation and analysis carried out by Snowy Mountains Engineering Corporation and Woodward Clyde showed that the factor of safety was low for the upstream slope above the first berm. However to model the observed behaviour of displacements at the surface and in an inclinometer which showed the development of a shear surface at a depth of about 11 m, it was necessary to model the cracks and softened zones which had formed in the core. These persisted at least down to full supply level. It was also necessary to model transient pore pressures which occur in the cracks/softened zones during rain events and pore pressures in the filters. Conventional effective stress analysis gave low factors of safety, but only for shallow surfaces through the rockfill, i.e. not modelling the real situations. The observed deformations could only be replicated by numerical modelling if very low moduli were used in the rockfill (in the order of 5 MPa) and weakening of the core.

During construction of remedial works, which included a rockfill berm on the upstream side and new filters and rockfill for the upper 10 m of the downstream side,

a wedge of the core was displaced nearly 50 mm upstream. This displacement also sheared an inclinometer installed from the dam crest at depths of up to 16 metres. Calculations showed that failure surfaces encompassing this wedge had a limit equilibrium factor of safety of about 1.3. Numerical analysis was consistent with this. What appears to have happened is that the relatively stiff core has sheared locally particularly during drawdown events at a number of depths within the section due to lack of support from the poorly compacted rockfill. Further details are given in Davidson *et al.* (2000).

11.6.3 The lessons learnt

The lessons learnt can be summarised as:

(a) Conventional effective stress analysis ignores pore pressures generated during shearing. This is conservative for over consolidated, dilative soils. As discussed in Section 6.1.3, Ladd (1991) shows that effective stress analysis of normally consolidated (and slightly over-consolidated) clays can very significantly overestimate the factor of safety. The error can be as much as 2 times.

(b) What must be recognised is that even though the soil in modern dams is given a degree of over consolidation by rolling and by partial saturation suction effects, this is often only equivalent in over-consolidation to the stress due to ≈20 m to 30 m of dam height so, in the lower part of larger dams, the dam core is likely to be normally consolidated. If the dam is affected by transient loading, such as drawdown or high reservoir levels or high pore water pressures in cracks in the crest (see below), the loading is of short duration compared to the drainage path, so undrained loading occurs and to model this correctly, either the pore pressures generated in shearing should be included in the effective stress analysis or undrained strengths used.

(c) For poorly compacted fills such as those in Hume No. 1 Embankment, which were wetted by seepage through the dam or foundation, the soils behave as normally consolidated clays even at shallow depths. In fact, the CPT data from adjacent to walls at Hume No. 1 Embankment and at O'Shannassy Dam shows that the fill may 'hang up' on the walls, so the effective vertical stress at depth is lower than that calculated by unit weight × depth to the surface and strengths are even lower than expected.

(d) For cases of marginal stability it is important to model the actual conditions carefully including using undrained strengths, cracks and softened zones in the partially saturated zone and pore pressures in cracks or softened zones during rain events. This of course means that site investigation methods capable of locating such cracks and transient pore pressures are needed, e.g. using trenching across the dam crest and using continuous reading or maximum pressure recording piezometers.

(e) One should be conscious of the possibility that large deformations of the core may occur where rockfill is poorly compacted, even though the factor of safety is well above 1.0. This is discussed in Hunter and Fell (2003d).

11.7 ANALYSIS OF DEFORMATIONS

11.7.1 Analyses of embankment cross sections

The analysis of deformations of embankment dams, either as an aid to the assessment of stability or to assess the likelihood of cracking or hydraulic fracture, has become possible within most dam engineering organizations, with the advent of powerful personal computers and relatively inexpensive finite element and finite difference analysis programs. The majority of the analyses carried out are for embankment cross sections and most of the literature is focused on these.

We do not propose to attempt to review these methods, and instead recommend that several overview and case-specific papers listed below be read. These include:

(a) *Overview – State of the art papers*:
- Eisenstein and Naylor in ICOLD (1986a) Static analysis of embankment dams.
- Duncan (1992, 1996a). State of the Art: Limit Equilibrium and Finite Element Analysis of Slopes. Duncan lists over 100 references of case studies.
- Duncan (1994). The role of advanced constitutive relations in practical applications.
- Vaughan (1994). Assumption, Prediction and Reality in geotechnical engineering.

(b) *Examples of strain softening*:
- Progressive failure and other deformation analyses; Potts *et al.* (1990, 1997), Muir-Wood *et al.* (1995), Dounias *et al.* (1988, 1996), Kovacevic *et al.* (1997).

Duncan (1996a) lists some of the uncertainties in numerical modelling of dams:

- Degree of compaction of the embankment materials – the actual may be different to that specified.
- Compaction water content – actual may be different to the specified.
- No test data – e.g. it is not practicable to test rockfill zones.
- Construction sequence and time.
- Simplified approximations of stress-strain behaviour – all constitutive relations are approximation of real behaviour.
- Reliability of field measurements.

He concludes: *"comparisons of the results of finite-element analyses with field measurements have shown that there is a tendency for calculated deformations to be larger than measured deformations. The reasons for this difference include: (1) soils in the field tend to be stiffer than soils at the same density and water content in the lab because of aging effects; (2) average field densities are higher than the specified minimum dry density, which is often used for preparing lab triaxial test specimens; (3) samples of in-place materials suffer disturbance during sampling, and are less stiff as a result; (4) many field conditions approximate plane strain, whereas triaxial tests are almost always used to evaluate stress-strain behaviour and strength; and*

(5) two-dimensional finite-element analyses over-estimate deformations of dams constructed in V-shaped valleys with steep valley walls.

By the time of this writing there has been about 30 years of experience with using the finite-element method to estimate stresses and deformations in slopes and embankments. This experience has shown the considerable potential of the method for use in engineering practice and it has pointed up clearly the sources of uncertainty in the results of these analyses. These are related primarily to difficulties involved in being able to predict the actual densities and water contents of soils in the field and with being able to anticipate the sequence of operations that will be followed during construction. With due allowance for the uncertainties, finite-element analyses afford a powerful method of analysis that is applicable to a wide variety of engineering problems."

Duncan (1996a) concluded (as had Vaughan, 1994, who discusses several cases), that there is much to be gained from the analyses in the understanding of mechanisms, if not the absolute stresses and deformations.

The authors' experience in this area is limited to that of 'user' rather than 'doer' of the analyses. From this it is clear that it is difficult to model the deformation of dams with any degree of precision. The ability to numerically model generally is greater than the ability to provide accurate material properties.

There are however often more basic reasons for poor modelling, including not 'constructing' (or numerically 'building') the slope in steps correctly, assuming drained conditions will apply when in fact undrained behaviour is actually present and not paying enough attention to the stiffness of the layers (or in particular relative stiffnesses), so the stress distributions and deformations are poorly modelled.

Few model pore pressures accurately or allow for cracking, softened zones around cracks, crack water pressures or in many cases, strain weakening. Given that these factors often control the stresses and deformations in slopes, it is not surprising the modelling is not good.

The authors' view is that the overwhelming problem in too many cases is that those doing the analysis have neither the proper understanding of the mechanics of the problem and what are reasonable properties, nor a good, detailed understanding of the computer programs. The result is an unfortunate waste of time and money.

The techniques are potentially valuable, done by competent persons, who understand dams, material behaviour and numerical analysis.

11.7.2 Cross valley deformation analyses

As discussed in Section 8.3.2 as an embankment dam is constructed the partially saturated compacted soil in the embankment consolidates and settlement occurs. Where the valley sides are steep and/or have steps in the profile differential settlements occur due to the variations in the height of the embankment, and these can lead to tensile or low stress zones in which cracks may form. Hydraulic fracture through these low stressed zones as the dam is filled or under flood conditions may also occur.

For most dams, about 80% to 90% of the total settlement occurs during construction (Hunter, 2003; Hunter and Fell, 2003d), so the stresses set up in the construction phase largely control the likelihood of low stress zones and cracking but post construction deformations are still important to the development of cracking.

It should be recognized that these stresses are inevitable even in well-designed and constructed dams and are not caused by poor design or construction practice.

There is often considerable benefit to be gained by carrying out cross valley numerical analyses of existing dams to determine the potential location and depth of low stress zones. These can be used to assist in assessing the likelihood of concentrated leak erosion as discussed in Section 8.3.

As for cross section analyses the results are dependent on the data inputs and are only a guide to performance. The most important parameters for such analyses are the Poisson's ratio and Young's moduli of the materials in the dam and/or the foundation if the dam is founded on a soil foundation.

Examples of these analyses are described in Section 8.3.2.

Modelling by Bui *et al.* (2004, 2005) showed that generally 2D analyses are sufficient. The exception is where the outer zones of the embankment have a much lower Young's modulus than the core; e.g. if a central core earth and rockfill dam is constructed with dumped rockfill. In these cases the deformations in the upstream/downstream direction (cross sectional deformation) are large and they lead to added settlement in the cross-valley direction. For these cases 3D analyses should be carried out.

11.8 PROBABILISTIC ANALYSIS OF THE STABILITY OF SLOPES

The use of probabilistic methods for assessing the stability of dam or other slopes, other than mine pit slopes, is not common. This is despite a large body of research and publication into the topic, which have brought the state of the art to a relatively high degree of sophistication.

G. Mostyn in Mostyn and Fell (1997) gives a thorough overview of the state of the art method. Other reviews are given in Christian *et al.* (1992) and Wu *et al.* (1996).

In the authors' experience where probabilistic analyses have been carried out for dams it has been a very costly exercise, with the results very dependent on the assumed parameters, particularly in regards to autocorrelation distances which are invariably guessed from limited published data. The results have been predictable in that where low factors of safety were being calculated high probabilities of failure resulted.

In another case the analyses were carried out in effective stress terms, and gave low probabilities of failure when in fact the soils were contractive on shearing and undrained parameters should have been used and would have more reliably modelled the actual behaviour, which was that the dam was deforming under high reservoir levels so was marginally stable.

Given the low incidence of slope instability failures of dams and the fact that many accidents and failures can be attributed to weaknesses in the dam (e.g. by softening) or the foundation (e.g. the presence of bedding surface shear in the rock), which would be missed in probabilistic analysis just as they were in the deterministic, factor of safety approach, the authors do not see a great need to use probabilistic analysis.

The exception could be for embankments built on soft clay foundations, where the method may help demonstrate that a considerable degree of conservation in selection of the factor of safety is required. See Duncan (2000) and Christian *et al.* (1992) for examples.

Chapter 12

Design of embankment dams to withstand earthquakes

12.1 EFFECT OF EARTHQUAKE ON EMBANKMENT DAMS

Earthquakes impose additional loads on to embankment dams over those experienced under static conditions. The earthquake loading is of short duration, cyclic and involves motion in the horizontal and vertical directions. Earthquakes can affect embankment dams by causing settlement and longitudinal and transverse cracking of the embankment, particularly near the crest of the dam. The settlements give a reduction of freeboard between the embankment crest and reservoir level which may, in the worst case, result in overtopping of the dam. Internal erosion and piping may develop in cracks. Liquefaction or loss of shear strength due to increase in pore pressures induced by the earthquake may occur in the embankment and its foundations. In short, earthquake loading may result in:

- Instability of the upstream and downstream slopes of the dam and large deformations.
- Differential settlements and cracking due to active faults passing through the dam foundation.
- Development of open cracks or opening of previously closed joints in the foundation, close to the core-foundation contact.
- Differential movement between the embankment, abutments and spillway structures leading to transverse cracks.
- Damage to outlet works passing through the embankment and differential settlements leading to cracking.

The potential for such problems depends on:

- The seismicity of the area in which the dam is sited.
- Foundation materials and topographic conditions at the dam site.
- The type and detailed construction of the dam.
- The water level in the reservoir at the time of the earthquake.

Earthquakes may also result in overtopping of the dam by seiches induced in the reservoir in the event of large tectonic movement in the reservoir and by waves due to earthquake induced landslides into the reservoir from the valley sides. These are not covered in this book.

The amount of site investigation, design and additional construction measures (over those needed for static conditions) will depend on these factors, the consequences of failure and whether the dam is existing or new.

There are three main issues to consider:

- The general (or "defensive") design of the dam, particularly the provision of filters, to prevent or control internal erosion of the dam and the foundation and provision of zones with good drainage capacity (e.g. free draining rockfill).
- Deformations induced by the earthquake (settlement, cracking) and resulting dam freeboard.
- The potential for liquefaction of saturated sandy and silty soils or gravels with a sand and silt matrix in the foundation and in the embankment and how this affects deformations and stability immediately after the earthquake.

The aim of this chapter is to present an overview of the topic, rather than to give detailed methods, because these are too complex and extensive to be covered here. The authors have drawn largely from several useful review papers including Seed (1979a, 1983), USNRC (1985), Finn (1993, 2000), Youd *et al.* (2001), Idriss and Boulanger (2008a, 2010) and Seed *et al.* (2003). Much of the description of earthquakes and their characteristics is taken from ANCOLD (1998) and was written by G. Gibson.

12.2 EARTHQUAKES AND THEIR CHARACTERISTICS

12.2.1 Earthquake mechanisms and ground motion

Earthquakes result when the stresses within the earth build up over a long period of time until they exceed the strength of the rock, which then fails and results in displacement along a fault.

The fault displacement in a particular earthquake may vary from centimetres up to many metres. Once ruptured, the fault is a weakness, which is more likely to fail in future earthquakes, so a large total displacement may build up from many earthquakes over a long period of time. This may eventually measure kilometres for thrust faults produced by compression, or hundreds of kilometres for strike-slip faults such as the San Andreas fault.

Figure 12.1 shows the types of fault and the effects on the ground near the fault.

The point on the fault surface where a displacement commences is called the focus or hypocentre and the earthquake epicentre is the point on the ground surface vertically above the focus. The displacement usually propagates along the fault in one direction from the focus, but sometimes it propagates in both directions. The centre of energy release is near but not exactly at the focus.

As shown in Figure 12.2 the focal distance from an earthquake to a point on the earth's surface is the three dimensional slant distance from the focus to the point, while the epicentral distance is the horizontal distance from the epicentre to the point.

Earthquake ground vibration is recorded by a seismograph or a seismogram. Modern seismographs record three components of motion: east-west, north-south and vertical.

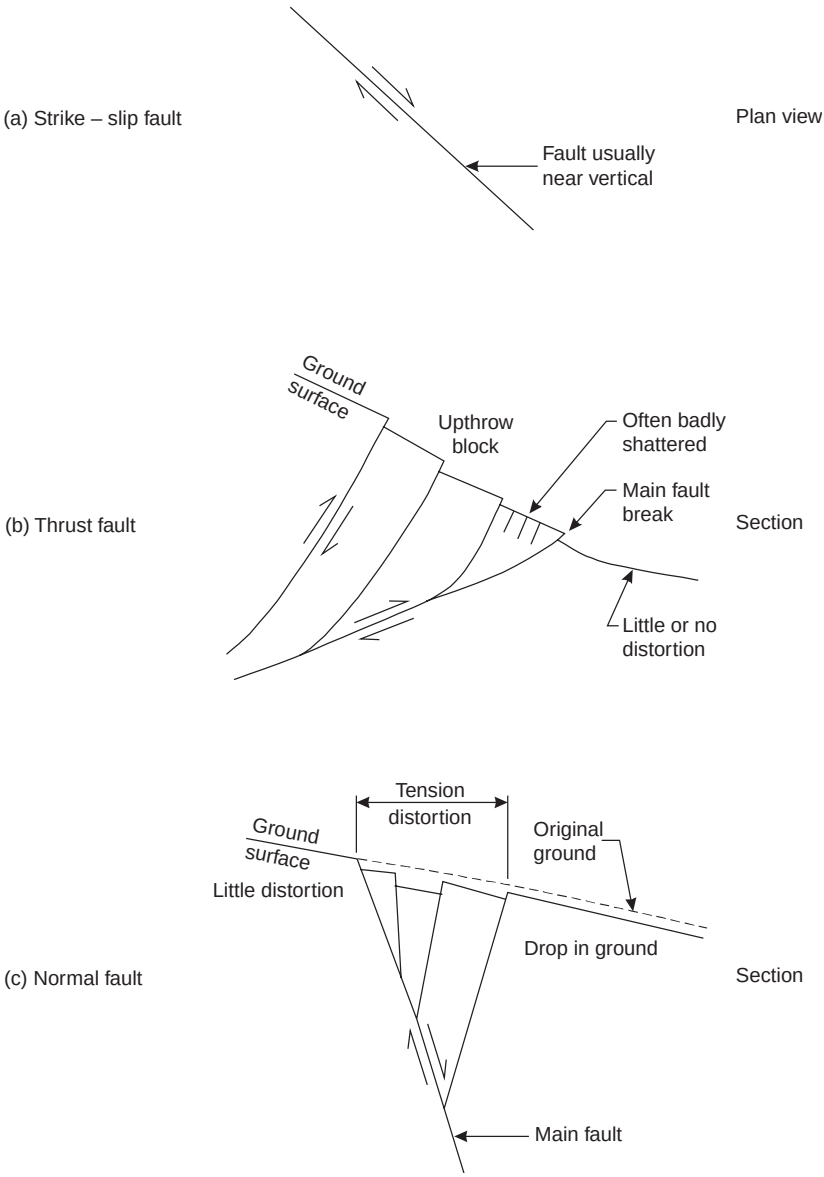


Figure 12.1 Schematic diagram of the types of fault and the effects on the ground nearby (adapted from Sherard *et al.,* 1974).

The rupture time for small earthquakes is a fraction of a second, for earthquakes of magnitude 5 it is about a second and for large earthquakes may be up to tens of seconds. However the radiated seismic waves travel at different velocities and are reflected and refracted over many travel paths, so the total duration of vibrations at a site persists longer than the rupture time and shows an exponential decay.

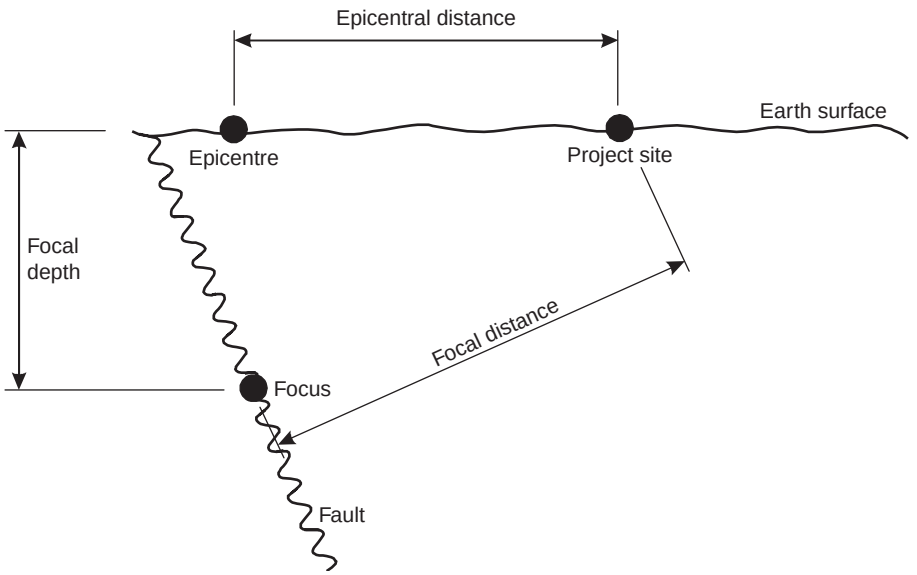


Figure 12.2 Definition of earthquake terms.

Several types of seismic wave are radiated from an earthquake. Body waves travel in three dimensions through the earth, while surface waves travel over the two-dimensional surface like ripples on a pond. There are two types of body wave (P and S waves), and two types of surface wave (Rayleigh and Love waves).

Primary or P waves are compression waves with particle motion parallel to the direction of propagation.

Secondary or S waves are shear waves, with particle motion at right angles to the direction of propagation. The amplitude of S waves from an earthquake is usually larger than that of the P waves.

P waves travel through rock faster than S waves, so they always arrive at a seismograph before the S waves.

The frequency content of earthquake ground motion covers a wide range of frequencies up to a few tens of hertz (cycles per second). Most engineering studies consider motion between about 0.2 and 25 Hz. For embankment dams, the lower frequency (0.5 to 5 Hz) motion corresponding to their natural frequency is most important.

The amplitude, duration and frequency content of earthquake ground motion at a site depend on many factors, including the magnitude of the earthquake, the distance from the earthquake to the site and local site conditions.

The larger the earthquake magnitude, the greater the amplitude, the longer the duration of motion and the greater the proportion of seismic energy at lower frequencies. A small earthquake has low amplitude (unless it is very close), short duration and has only high frequencies.

The smaller the distance from an earthquake to the site, the higher the amplitude. The duration is not strongly affected by distance. High frequencies are attenuated

by absorption within the ground more quickly than low frequencies, so at greater distances the proportion of seismic vibration energy at high frequencies will decrease.

12.2.2 Earthquake magnitude and intensity

There are a number of different measures of the magnitude of an earthquake.

(a) *The Local or Richter magnitude*

$$M_L = \log_{10} A \tag{12.1}$$

where A = maximum seismic wave amplitude (in thousandths of a millimetre) recorded from a standard seismograph at a distance 100 km from the earthquake epicentre or

$$M_L = \log_{10} A - F(\Delta) + k \tag{12.2}$$

where $F(\Delta)$ = distance correction; k = scaling constant.

This allows calculation of M_L at different distances from the earthquake.

(b) *The Body Wave magnitude*

$$M_b = \log_{10} V + 2.3 \log \Delta \tag{12.3}$$

where V = maximum ground velocity in microns/sec recorded by the seismograph; Δ = distance from epicentre in kilometres.

(c) *The Moment Magnitude* (M_W) which assigns a magnitude to the earthquake in accordance with its seismic moment M_O, which is directly related to the energy released by the earthquake:

$$M_W = (\log_{10} M_O/1.5) - 10.7 \tag{12.4}$$

where M_O is the seismic moment in dyn-cm.

Moment magnitude is the scale most commonly used for engineering applications. Figure 12.3 shows the relationship between the different scales.

Earthquakes with magnitude of less than 3 or 4 will usually not cause any felt effect, and earthquakes with a magnitude less than about 5 will usually not cause any damage unless they are shallow, near the structure and/or ground conditions cause amplification. The maximum recorded magnitude is approximately 8.9. The scale is not linear. Each step in the magnitude scale represents a thirty-fold increase in energy released by the earthquake, e.g. a Magnitude 5 earthquake represents 900 times the energy of a Magnitude 3 and a Magnitude 8 represents about 10^6 times the energy of a Magnitude 4.

Another measure of earthquake size is the fault area, or the area of the fault surface which is ruptured. The fault area ruptured in an earthquake depends on the magnitude

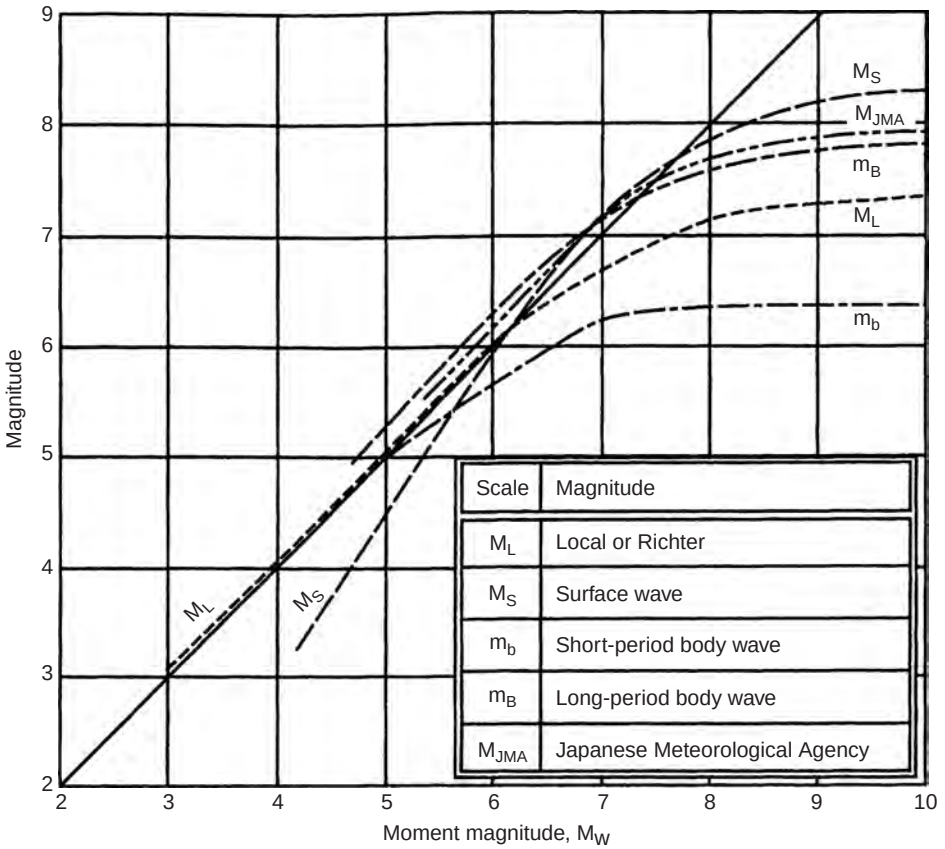


Figure 12.3 Relationship between Moment, M_W and other magnitude scales (Youd and Idriss, 1997; after Heaton *et al.*, 1982).

and stress drop in the earthquake. For a given magnitude, a higher stress drop will give a small rupture area. Typically, a Magnitude 4 earthquake ruptures a fault area of about 1 square kilometre, Magnitude 5 about 10 square kilometres, and Magnitude 6 about 100 square kilometres.

Earthquake intensity is a qualitative value based on the response of people and objects to the earthquake. The intensity depends on distance from the earthquake, ground conditions and topography, so there will be a range of intensity values for any earthquake. The most commonly used scale is the Modified Mercalli Scale which is reproduced in Table 12.1.

For the design of dams, the horizontal and vertical acceleration induced by the earthquake at the base of the dam are usually required. Information is best obtained from accelerograph measurements at the dam site, but in many cases will be obtained from records of sites with similar geological conditions. A typical accelerograph record is shown in Figure 12.4.

Table 12.1 Modified Mercalli Scale, 1956 version (Richter, 1958; Hunt, 1984).

Intensity	*Effects*
I	Not felt. Marginal and long period effects of large earthquakes.
II	Felt by persons at rest, on upper floors, or favourably placed.
III	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.
IV	Hanging objects swing. Vibration like passing of heavy trucks or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wood walls and frames creak.
V	Felt outdoors, duration estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swings, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.
VI	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc. off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle – CFR).
VII	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments – CFR). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Steering of motor cars affected. Damage to masonry C, partial collapse. Some damage to masonry B, none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations – CFR). Frame structures, if not bolted, shifted off foundations. Frames cracked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.
X	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beachheads and flat land. Rails bent slightly.
XI	Rails bent greatly. Underground pipelines completely out of service.
XII	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

Note: Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).
– Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces;
– Masonry B: Good workmanship and mortar; reinforced, but not designed to resist lateral forces;
– Masonry C: Ordinary workmanship and mortar; no extreme weaknesses such as non-tied-in corners, but masonry is neither reinforced nor designed against horizontal forces;
– Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally;
– CFR indicates additions to classification system by Richter (1958).

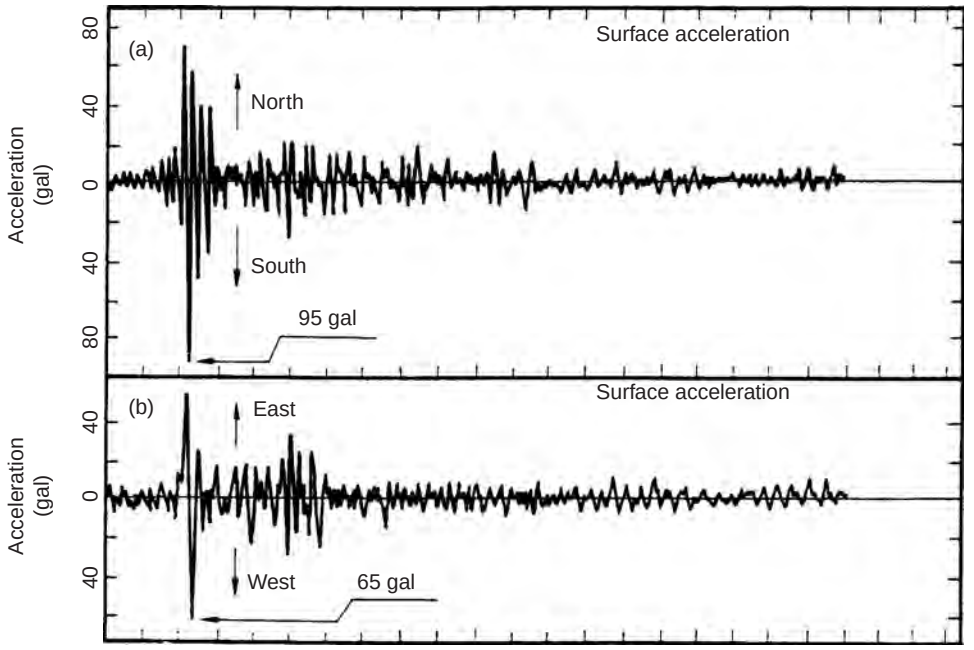


Figure 12.4 Earthquake accelerograph (USNRC, 1985).

12.2.3 Attenuation and amplification of ground motion

Earthquake ground motion attenuates with increasing distance from the source due to radiation and hysteretic damping. High frequency motion is attenuated more quickly with distance than lower frequency motion.

For estimates of peak ground acceleration, attenuation is allowed for by using an attenuation function of the form:

$$a = b_1 e^{b_2 M} R^{-b_3} \tag{12.5}$$

where a = acceleration; R = focal distance; M = Magnitude; b_1, b_2, b_3 are constants, which vary considerably over the world.

Some earthquake hazard studies use the Esteva and Rosenblueth (1969) attenuation functions, which give peak ground velocity (mm/s), peak ground acceleration (mm/s^2) at an epicentral distance × kilometres from an earthquake at focal depth z kilometres with local magnitude M_L. The equations are:

$$R = \sqrt{x^2 + z^2 + 400} \tag{12.6}$$

$$v_{peak} = 160\, e^{1.0 M_L} R^{-1.7} \tag{12.7}$$

$$a_{peak} = 20000\, e^{0.8 M_L} R^{-2.0} \tag{12.8}$$

Because of the "400" term in the expression for R, corresponding to a minimum R of 20 kilometres, these equations give low values of ground motion at distances closer than a few kilometres.

There have been many different attenuation relationships developed around the world. These are sometimes specific to a tectonic domain. Recently, updated ground motion models, such as the NGA (Next Generation Attenuation) models (e.g. Abrahamson *et al.*, 2008) have been developed that quantify site characteristics in a much more rigorous way than earlier models. These models specify the site characteristics using Vs30, which is the average shear wave velocity in the uppermost 30 metres below the ground surface.

The attenuation equations discussed above, allow estimation of bedrock ground motions, e.g. peak ground acceleration (pga), at a site.

As discussed by Martin (1988), sites underlain by stiff soil and thick cohesionless soil may experience similar peak accelerations to the "bedrock" values. However, in sites underlain by soft or medium soft clays, or shallow cohesionless soils, the maximum ground acceleration may be quite different from bedrock accelerations. Martin (1988) indicates that for ground accelerations less than 0.1 g amplification may occur, whereas for very high accelerations (e.g. >0.3 g) the peak accelerations may be attenuated (de-amplified) due to energy dissipation. The authors' experience is that, even for stiff and thick cohesionless soil, amplification may be calculated to occur for small earthquakes. Amplification effects can be estimated using computer programs such as SHAKE or DESRA, using input ground motions from typical accelerograms.

Topographic effects are also important. An accelerograph sited on a ridge high on a dam abutment, is likely to record significantly higher accelerations than one sited at the river level, so care must be taken in locating such instruments. At Pacoima Dam, California, in 1971, an instrument on top of one of the abutments recorded a pga over 1.2 g on rock; the estimated pga at the base of the dam was 0.6 g.

12.2.4 Earthquakes induced by the reservoir

As discussed in ICOLD (1983, 1989c, 2012), there is documented evidence to prove that impounding of a reservoir sometimes results in an increase of earthquake activity at or near the reservoir.

ICOLD (1983) conclude that:

- Earthquakes of magnitude 5 to 6.5 were induced in 11 of 64 recorded events.
- The greatest seismic events have been associated with very large reservoirs (but there is insufficient data to show any definite correlation between reservoir size and depth and seismic activity).
- In view of the above, a study of possible induced seismic activity should be made at least in cases where the reservoir exceeds 5×10^8 m^3 in volume, or 100 m in depth.
- The load of the reservoir is not the significant factor; rather it is the increased pore water pressure in faults, leading to a reduction in shear strength over already stressed faults.

ICOLD (2012) indicate that there are so far only six generally accepted cases of reservoir triggered seismicity where the magnitude of the event exceeded 5.7. The largest recorded magnitude event that is believed to be due to a reservoir-triggered event is 6.3.

ANCOLD (1998), as advised by G. Gibson, indicates that reservoir induced seismicity events usually occur initially at shallow depth under or immediately alongside a reservoir. As years pass after first filling and groundwater pore pressure increases permeate to greater depths and distances, the events may occur further from the reservoir. This occurs at a rate of something like one kilometre per year.

Reservoir induced seismicity is experienced under new reservoirs, usually starting within a few months or years of commencement of filling and usually not lasting for more than about twenty years. Once the stress field and the pore pressure fields under a reservoir have stabilized, then the probability of future earthquakes reverts to a value similar to that which would have existed if the reservoir had not been built. Most of the earthquake energy does not come from the reservoir, but from normal tectonic processes. The reservoir simply acts as a trigger.

If there is a major fault near the reservoir, reservoir induced seismicity can produce earthquakes exceeding Magnitude 6.0 (Xinfengjiang, China, 1962, M6.1; Koyna, India, 1967, M6.3). Such events will occur only if the fault is already under high stress. A number of Australian reservoirs have triggered earthquakes equalling or exceeding Magnitude 5.0 (Eucumbene, 1959, M5.0; Warragamba, 1973, M5.0; Thomson, 1996, M5.2).

It is more common for a reservoir to trigger a large number of small shallow earthquakes, especially if the underlying rock consists of jointed crystalline rock like granite (Talbingo, 1973 to 1975; Thomson, 1986 to 1995). These events possibly occur on joints or local minor faults rather than major faults, so are limited in size and only give magnitudes up to 3 or 4. There is no hazard to the dam from such low magnitude reservoir induced earthquakes, even if they occur regularly. Their shallow depth means that they may often be felt or heard.

12.3 EVALUATION OF SEISMIC HAZARD

12.3.1 Terminology

The following terminology is based on ANCOLD (1998), and ICOLD (2012).

Operating Basis Earthquake – the OBE is that level of ground motion which will cause only minor and acceptable damage to the dam and appurtenant structures. The dam, appurtenant structures and equipment should remain functional and damage from the occurrence of earthquake shaking not exceeding the OBE should be easily repairable.

Safety Evaluation Earthquake – the SEE is the maximum level of ground motion for which the dam should be designed or analysed. Damage can be accepted but there should be no uncontrolled release of water from the reservoir. The term *Safety Evaluation Earthquake* replaces the term *Maximum Design Earthquake* (MDE) in ICOLD and ANCOLD.

Maximum Credible earthquake The MCE is the largest reasonably conceivable earthquake magnitude that is considered possible along a recognized fault or within

a geographically defined tectonic province, under the presently known or presumed tectonic framework.

Maximum Credible Earthquake Ground motion. The most severe ground motion affecting a dam site due to an MCE.

The OBE and SEE is expressed in terms relevant to the design of the dam or appurtenant structure. It may be in terms of peak ground acceleration and moment magnitude Mw, peak ground velocity or response spectra.

12.3.2 General principles of seismic hazard assessment

12.3.2.1 *Probabilistic approach*

This method is generally used where active faults cannot be identified or to assess the hazard from earthquakes in areas away from recognised active faults. The probabilistic estimation of ground motion requires the following seismicity information about the surrounding area:

- The rate of occurrence and magnitude of earthquakes.
- The relative proportion of small to large events (b value).
- The maximum earthquake size expected (maximum credible magnitude).
- The spatial distribution of earthquake epicentres including delineation of faults.

The seismicity can be evaluated by a modified Richter relation (Gibson, 1994).

$$\log_{10}(P) = -\log_{10}[10(10^{-bM} - 10^{-bM_{max}})] - \log_{10}(N_o) \tag{12.9}$$

where P is the return period in years for an earthquake of magnitude M or greater and N_o is the rate of occurrence of earthquakes, given as the number of earthquakes of Magnitude zero or greater per year per unit volume or per unit area. An area of 100×100 kilometres is commonly used. Data from this area must be converted to per square kilometre or per cubic kilometre for ground motion calibrations.

"b" is the Richter "b" value, which gives the relative number of small earthquakes to large. It is the logarithm to the base 10 of the ratio of the number of events exceeding $M + 1$ to the number exceeding M. A value of 1.0 would correspond to ten times as many earthquakes exceeding magnitude M as would exceed magnitude $M + 1$.

M_{max} is the magnitude of the maximum credible earthquake for the area. Because of the low probability of very large earthquakes, M_{max} does not critically affect ground motion recurrence estimates for return periods up to hundreds of years, especially when the "b" value is high. However, it is more important for low AEP events such as may be important for design of high hazard dams. That maximum credible magnitude causes the magnitude recurrence plot to flatten out and asymptote to that value.

The seismicity parameters N_o and b are determined using available earthquake data. In most places there are insufficient earthquake data to determine M_{max} from historical records, but values can be estimated by considering the tectonic situation and local fault dimensions. The "b" value is much more critical than N_o and a small adjustment to "b" will give a large change in predictions.

Hazard evaluation depends on the extrapolation of data from small earthquakes to larger magnitudes. The lower the "b" value, the greater the resulting hazard estimate.

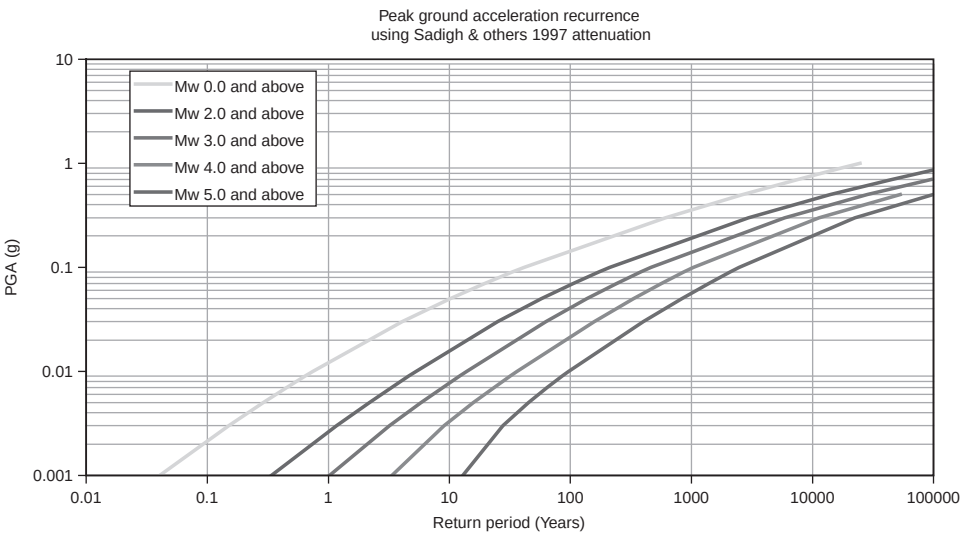


Figure 12.5 Peak ground acceleration versus annual exceedance probability (AEP) and earthquake magnitude for a typical site in Australia.

A catalogue with missing small earthquakes will give an invalid low "b" value and an estimated hazard that will be too high. A catalogue with smaller quarry blasts incorrectly identified as earthquakes will give a high "b" value and an extrapolation that is non-conservative.

This information is used with one or more attenuation coefficients to determine ground motions at the dam site.

Figure 12.5 shows the output of a probabilistic assessment of seismic hazard expressed in peak ground (bedrock) acceleration – as would be required for analysis of liquefaction. Note that the contribution of earthquake magnitude is separated, because the magnitude, as well as peak ground acceleration is important. Figure 12.6 shows an example of seismic response spectra and Figure 12.7 the magnitude-distance contributions which are important when assessing liquefaction and deformations.

12.3.2.2 *Seismic hazard from known active or capable faults*

This method is used where active faults or faults capable of being active in the vicinity of the dam can be identified. The procedure will usually involve:

(a) Identification of major faults within the vicinity of the dam. This may involve an area up to several hundred kilometres from the site. Figure 12.8 shows an example.
(b) Assessment of whether the faults are active or potentially active, by consideration of whether modern (including small) earthquakes have been recorded along the fault. This may also involve geomorphological studies, e.g. of displaced

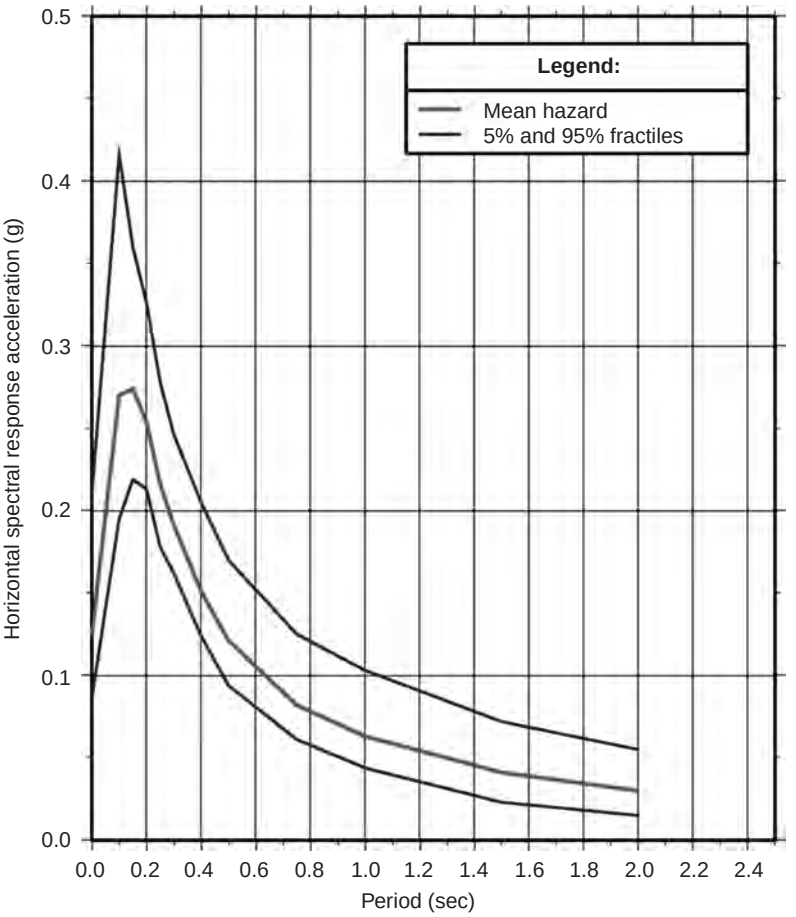


Figure 12.6 Example of seismic response spectra showing uncertainty.

river terraces and/or trenching across faults to identify past displacements and determine their ages.

(c) Assessment of the Maximum Credible Earthquake magnitude on each identified fault. This will usually be determined by considering the length and/or area of the fault and the type of fault. The likely focal depth and, hence, focal distance are also estimated.

(d) For a deterministic approach, assess the pga and/or seismic response spectra at the project site resulting from the MCE at each of the faults using appropriate attenuation coefficients and determine the most critical earthquake.

The duration of the earthquake and the period of oscillations are dependent on the magnitude of the earthquake and the effect on the dam is dependent on these factors as well as maximum accelerations, therefore more than one earthquake may have to be used in the analyses.

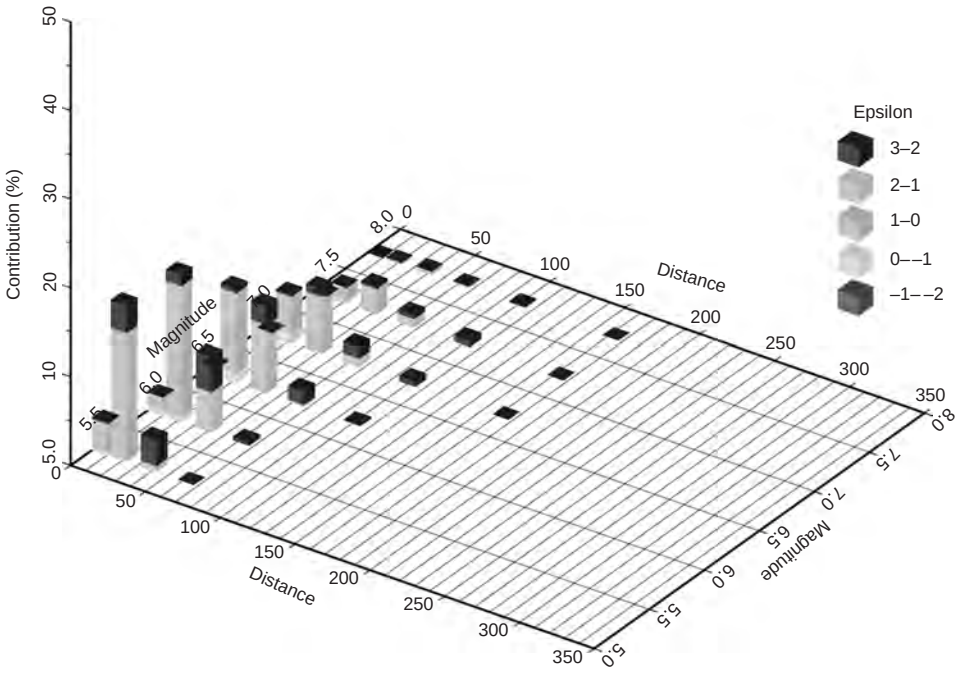


Figure 12.7 Example of magnitude-distance contributions to the seismic hazard.

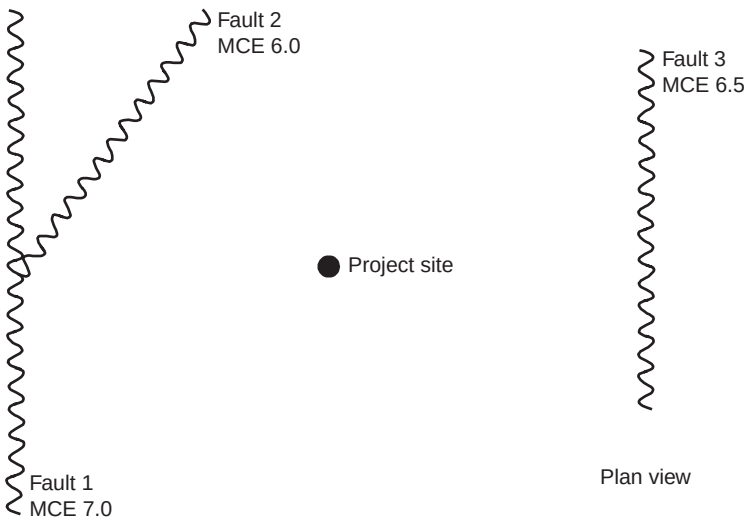


Figure 12.8 Plan of a project site and potentially active faults for assessment of seismic hazard.

(e) For a risk based approach, determine the annual probability versus earthquake magnitude for each fault. For example for the situation shown in Figure 12.8.

Fault	Annual exceedance probability	Earthquake magnitude
1	0.01	5.5
	0.001	6.5
	0.0001	7.0(MCE)
2	0.01	5.0
	0.001	6.0(MCE)
3	0.01	5.5
	0.001	6.5(MCE)

From this estimate pga versus combined annual exceedance probability and seismic response spectra using appropriate attenuation coefficients, allowing for the three faults and accounting for earthquake magnitude.

In practice probabilistic approaches should combine the method outlined in Section 12.3.2.1 with allowance for active faults.

USNRC define a capable fault as a fault which has exhibited one or more of the following characteristics:

(a) Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.
(b) Macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
(c) A structural relationship to an active fault according to characteristics (a) or (b) that movement on one could be reasonably expected to be accompanied by movement on the other.

ICOLD (2012) indicates that in a deterministic procedure for choosing seismic evaluation parameters, the critical active faults which show evidence of movements in Holocene time (i.e. in the last 11,000 years), large faults which show evidence of movements in Latest Pleistocene time (i.e. between 11,000 and 35,000 years ago) or major faults which show evidence of repeated movements in Quaternary time (1.8 million years) should be considered. The capability of these faults must be ascertained through appropriate established methods such as a rupture length-magnitude relationship or a fault movement and magnitude relationship.

12.3.3 Other forms of expression of seismic hazard

The discussion so far has centred on assessing the hazard in peak ground acceleration (on bedrock) terms.

For more advanced dynamic analysis, it will be necessary to obtain accelerograms of earthquakes of a similar nature to that expected considering the source faults, scaled or spectrally matched to the expected peak ground accelerations. In some cases, synthetic accelerograms may be used. More than one accelerogram will usually be needed. Suitable time histories should be selected in consultation with the project seismologist.

For analysis of concrete gravity dams and other structures response spectra may be required.

12.3.4 Selection of design seismic loading

There are two ways of selecting the design seismic loading:

12.3.4.1 *Deterministic approach*

Operating Basis Earthquake (OBE)

The selection of the OBE is a matter for the Owner and other stakeholders such as the water supply authority which relies upon the dam, to consider in consultation with the Consultant. The selection should take account of the criticality of the dam and the water it supplies.

The OBE is often accepted as a loading which has a 10% chance of being exceeded in a 50 year period, or an annual probability of exceedance of 1 in 475.

Safety Evaluation Earthquake (SEE)

The SEE is selected depending on the Consequence Category of the dam. The authors suggest the loadings shown in Table 12.2.

Table 12.2 Suggested seismic design loads.

Dam Consequence Category	*Operating Basis Earthquake (OBE)*	*Safety Evaluation Earthquake (SEE)*
Extreme and High Consequence Category Dams	Owner to determine. Commonly 1 in 475 AEP up to 1 in 1,000 AEP	The greater of: Loading from MCE on known active faults (85th fractile) and Probabilistic loading[4]: Extreme and High A: 1 in 10,000 AEP High B: 1 in 5,000 AEP High C: 1 in 2,000 AEP
Significant Consequence Category Dams	Owner to determine. Commonly 1 in 475 AEP	The greater of: Loading from MCE on known active faults (50th fractile) and Probabilistic loading[4]: 1 in 1,000 AEP
Low Consequence Category Dams	Owner to determine. Commonly 1 in 475 AEP	Probabilistic loading[4]: 1 in 1,000 AEP

Notes:
(1) A factor of safety should be applied to estimated stresses and deformations within the dam and its foundations to give a low likelihood of the dam failing given the SEE loading.
(2) Active faults are as defined in Section 12.1.
(3) Owner and other stakeholders to determine in consultation with the consultant.
(4) Median, 50th fractile.
(5) Loads to be applied to the dam with reservoir at full supply level.

Table 12.2 is based on ICOLD (2012), ANCOLD (2012), and Canadian Dam Safety Association (2007). Tables 10.3 and 10.4 define Consequence Categories according to ANCOLD (2012), see that publication for details of how to estimate damages and losses.

Note (1) in Table 12.2 says "*A factor of safety should be applied to estimated stresses and deformations within the dam and its foundations to give a low likelihood of the dam failing given the SEE loading*". As a guide to what may be required, to satisfy ANCOLD (2003) tolerable risk criteria would mean that the conditional probability of the dam failing given the SEE would be less than 1 in 100.

As there is no unique relationship between factor of safety and likelihood of failure judgement has to be applied. It will be useful to consider the effect on the dam of the 85th fractile and 95th fractile loads. These are only about 3 times and 10 times less likely respectively than the median or 50th fractile load. If the dam would almost certainly fail leading to breach for either of these loads the likelihood of failure would be too high to satisfy ANCOLD (2003) tolerable risk guidelines.

If the MCE loading is significantly larger than the probabilistic loading detailed discussions should be held with the seismologist to better understand the characteristics of the active fault and further investigations carried out if required. Once the MCE loading is finalised either adopt it or/and adopt a risk based rather than deterministic approach.

If there is some doubt about whether a fault is active or not, a suggested approach is to consult the seismologist(s) involved and assign a probability that the fault is active and apply this in the event tree logic in the risk analysis.

For Significant and Low Consequence Category dams, if the structure is susceptible to liquefaction or has components which will fail at loads only a little greater than the loads in Table 12.2, check the design for the critical load and assess the adequacy of the design using risk assessment methods.

For some concrete dams the reservoir partly full or empty design case may be more critical than reservoir at full supply level. If so consider this case allowing in the risk analysis for the likelihood the reservoir will be lower than full supply level at the time of the earthquake.

12.3.4.2 *Risk based approach*

For risk based assessments the seismic loads should cover the complete range of feasible loads.

For the risk analysis the seismic loads should be partitioned taking account of the fragility of the structure. For example:

(i) If liquefaction is an issue, the earthquake loading with an AEP at which liquefaction is widespread would be used as a partition boundary.
(ii) The AEP of the earthquake loading which results in critical retaining walls failing, e.g. between spillway and embankment.
(iii) The AEP of the earthquake loading which is likely to result in displacement of a concrete gravity dam.

12.3.4.3 *Which approach to use?*

National codes, or guidelines to practice, will often specify which method should be used.

The authors favour the risk based approach for the following reasons:

1. In the deterministic approach it is difficult to account for the fact that the conditional probability of failure of the dam given the SEE is not 1, and may be very different for different types of dams and foundation conditions. Hence either these conditional probabilities have to be estimated, or it has to be understood that the different dams have quite different annual probabilities of failure, even though they are assessed for the same SEE.
2. Deterministic methods potentially mask the fact that many dams have some likelihood of failure at loads less than the SEE.
3. Some design methods are by their nature risk based; e.g. liquefaction assessments.
4. Risk based methods are commonly used for Portfolio Risk Assessments (PRA) in some countries such as Australia, and if deterministic methods are used they may result in a dam passing the deterministic method but not satisfying the PRA tolerable risk criteria.
5. It is relatively easy to allow for the uncertainty in the estimation of the seismic hazard.

Whichever approach is taken, the bedrock ground motions need to be adjusted where appropriate for amplification (or de-amplification) effects.

12.3.5 Modelling vertical ground motions

Gulerce and Abrahamson (2011) provide ground-motion prediction equations (GMPEs) for the vertical-to-horizontal spectral acceleration (V/H) ratio, and the methods for constructing vertical design spectra that are consistent with the probabilistic seismic hazard assessment results for the horizontal ground motion component.

The proposed V/H ratio GMPE is dependent on the earthquake magnitude and distance, and accounts for the differences in the nonlinear site-response effects on the horizontal and vertical components. This results in large V/H ratios at short spectral periods for soil sites located close to large earthquakes.

It is suggested that this method be used rather than using a constant ratio of vertical to horizontal ground motion as has been commonly done.

12.3.6 The need to get good seismological advice

Seismic hazard should be assessed by a seismologist familiar with the seismic hazard in the project area and experienced in assessing seismic hazard for dams.

The following outlines some of the requirements of a seismic hazard assessment:

For High and Extreme Consequence Category dams the following features should be included:

(a) Probabilistic analyses based on seismic source zones appropriate for the area of the study. There will sometimes be more than one model which is considered

appropriate in which case it is best both models are used. The weighting between these should be determined by the seismologist in consultation with the Owner and Consultant.

(b) Active faults which could influence the seismic hazard for the dam should be identified and should be accounted for in the seismic hazard assessment.

(c) Information on any known active faults in the vicinity of the dam and in particular those which have the capability to cause displacement in the foundation of the dam appurtenant structures or reservoir rim should be provided.

(d) Modelling should use more than one attenuation model. These may be specific for the area but if none is available at least one of the coefficients should be NGA. Weightings applied to the models should be determined by the seismologist in consultation with the Owner and Consultant.

(e) Shear wave velocity of the foundation rock should be determined and included in the attenuation modelling. The shear wave velocity may vary from valley to abutment sections so the seismic hazard may be different in the valley and the abutments. Amplification due to sediments overlying the rock should be modelled by the Consultant and is not part of the seismic hazard assessment.

(f) The response spectra used for the hazard assessment should be designed to allow for the response of the dam and appurtenant structures at the dam. Conditional mean spectra may be required, preferably more than just one response spectrum for the dam site.

(g) The hazard assessment should be carried out for earthquake magnitudes Mw of 4 or more. The results should be presented in a manner that the Consultant can assess the hazard for Mw > x. For embankment and concrete dams and for liquefaction assessments only Mw > 5.25 should be included. For some appurtenant structures earthquakes of smaller magnitude may be included.

(h) Uncertainty in the inputs and models should be modelled. The report should include plots of median (50th fractile), 85th fractile and 95th fractile plots. The report should also include plots of earthquake magnitude – distance contributions.

(i) Guidance on how to select time-history motions (accelerograms) suited for the dam and the appurtenant works, including appropriate selection of scaled or spectrally matched time histories.

(j) For new dams, guidance on whether reservoir induced seismicity need be considered and, if so, the resultant seismic loading.

For Significant and Low Consequence Category Dams, Owners may opt with the agreement of their Consultant to not require items (2), (3), and (10); use estimated shear wave velocities (5); and provide mean estimates without modelling uncertainty (8). However if that study shows that the seismic hazard is critical to the assessment of the risks posed by the dam the more complete assessment will be required.

For low consequence category dams it may be appropriate to conduct an initial assessment based on existing PSHA from nearby dams. Depending on the dam and its characteristics a decision can then be made as to whether a site specific PSHA is required.

It should be noted that PSHA become dated as new methods are developed and data bases improved. It is unlikely that a PSHA more than about 5 years old will be

reliable. The advice of the seismologist who carried out that study should be sought to advise whether a new study is warranted.

12.4 PRINCIPLES OF RISK BASED ANALYSES FOR EARTHQUAKE LOADS

12.4.1 General principles

The steps involved in this process are:

(i) Determine the AEP of earthquake ground motion (P_E) over the range of earthquake events which may affect the dam. Table 12.3 gives an example for AEP vs ground acceleration.
(ii) Determine the conditional probability (P_{BC}) that the dam will breach for each of the ground motion ranges (e.g. 0.125 g to 0.175 g in Table 12.3). In assessing this conditional probability all modes of failure should be considered and the probabilities combined, making allowance for interdependence and mutual exclusivity or otherwise (e.g. for embankment dams slope instability, piping, liquefaction/instability, and for concrete gravity dams overturning and sliding).
(iii) Assess the probability of failure for each range of ground motion by multiplying the AEP with P_{BC} i.e. $P_B = P_E \times P_{BC}$ – see Table 12.3.
(iv) Sum the probabilities to give the overall annual probability of failure due to earthquake.

This assessment should allow for modelling the uncertainties in estimating the seismic hazard, the likelihood of liquefaction, the liquefied strength, and deformations which may result. These are by their nature very approximate.

The extent to which this is modelled will depend on the consequence category of the dam, and whether the outcomes of the assessment are clear cut or marginal. These are readily modelled if a risk based approach is being followed.

Table 12.3 Example of assessing the probability of failure by earthquake.

Acceleration	*Annual probability (P_E)*	*Conditional[1] probability (P_{BC})*	$P_B^{(2)}$
<0.075 g	0.874	0.0005	0.0004
0.075 g to 0.125 g	0.100	0.005	0.0005
0.125 g to 0.175 g	0.015	0.05	0.0007
0.175 g to 0.225 g	0.007	0.1	0.0007
0.225 g to 0.3 g	0.003	0.3	0.0009
>0.3 g	0.001	0.5	0.0005
Total	1.000		0.0037

[1] Given the earthquake occurs.
[2] $P_b = P_E \times P_{BC}$.

12.4.2 Failure by loss of freeboard and overtopping

When assessing the likelihood of failure of an embankment dam by loss of freeboard due to seismic loading consider:

1. The likelihood that the soils are subject to liquefaction or cyclic softening.
2. The probability, given the earthquake loading that liquefaction occurs.
3. Given liquefaction occurs, whether the crest settles sufficiently to lose freeboard, taking account of the reservoir level at the time of the earthquake.
4. Given liquefaction does not occur, whether the crest settles sufficiently to lose freeboard taking account of the reservoir level at the time of the earthquake.
5. Given freeboard is lost, whether breach occurs.
6. Do this for both the upstream and downstream slopes of the embankment and for the varying geotechnical conditions which may apply over the length of the embankment and its foundations.

12.4.3 Failure by cracking and internal erosion and piping

When assessing the likelihood of failure for cracking under earthquake loading leading to internal erosion and piping consider:

1. The probability, given the earthquake, settlement occurs resulting in transverse cracking.
2. Given it occurs, whether it will persist to below the reservoir level at the time of the earthquake or before repairs can be carried out.
3. Given it does, whether erosion will initiate along the crack, whether filters will prevent erosion continuing, whether erosion progression and a breach forms.

This is seldom a dominant failure mode except in areas of high seismic hazard because "normal" and "flood" load conditions tend to dominate internal erosion and piping failure modes. It should be noted that dams which are susceptible to cross valley differential settlement may not require large earthquakes to crack.

12.5 LIQUEFACTION OF DAM EMBANKMENTS AND FOUNDATIONS

12.5.1 Definitions and the mechanics of liquefaction

12.5.1.1 *Definitions*

The following definitions are based on USNRC (1985), Robertson and Fear (1995), Robertson and Wride (1997), Lade and Yamamuro (1997) and Fell *et al.* (2000).

"*Liquefaction*" All phenomena giving rise to a reduction in shearing resistance and stiffness and development of large strains as a result of increase in pore pressure under cyclic or monotonic (static) loading of contractive soils.

"*Initial liquefaction*" is the condition when effective stress is momentarily zero during cyclic loading.

"*Flow liquefaction*" is the condition where there is a strain weakening response in undrained loading and the *in situ* shear stresses are greater than the steady state undrained shear strength.

"*Temporary liquefaction*" is the condition where there is a limited strain weakening response in undrained loading; at larger strain the behaviour is strain hardening. It is important to recognize that these phenomena apply to monotonic (static) as well as cyclic loading and are apparent in contractive soils, both cohesionless and those with some clay content.

Robertson and Fear (1995) and Robertson and Wride (1997) define two further terms which apply to cyclic loading.

"*Cyclic liquefaction*" is a form of temporary liquefaction, where the cyclic loading causes shear stress reversal and an initial liquefaction (zero effective stress) condition develops temporarily.

"*Cyclic mobility*" is a form of temporary liquefaction where the shear stresses are always greater than zero.

Idriss and Boulanger (2008) define "*Cyclic softening*" as a term used to describe the reduction of shear strength and stiffness of clays and plastic silts under cyclic loading.

These are discussed further below.

12.5.1.2 *Some consideration of the mechanics of liquefaction of granular soils*

Idriss and Boulanger (2008) give an overview of the fundamentals of liquefaction behaviour in Section 2. It is recommended that all those involved in liquefaction assessments read this overview.

USNRC (1985) also provides useful background on the development of methods for assessing liquefaction.

The following discussion is meant to give readers a basic understanding of the mechanics of liquefaction.

12.5.1.2.1 *Undrained shear of saturated granular soils*

Figure 12.9 shows schematically the undrained behaviour of saturated sand in triaxial compression.

A soil which has an initial void ratio higher than the ultimate state (steady state, critical state/line, point "SS" on Figure 12.9) will tend to contract (densify) in drained loading, reaching a void ratio equal to that at the ultimate state (critical state) line at that mean effective stress. In saturated undrained loading the soil is unable to contract, so positive pore pressures are developed, which will give strain weakening behaviour, reaching the steady state undrained strength S_{us}. If the static gravitational shear stresses q_{st} are greater than S_{us}, flow liquefaction may result.

A soil which has an initial void ratio just above the ultimate state line (e.g. point "LSS" on Figure 12.9) may show limited strain softening, or temporary liquefaction, with the soil wanting to contract on initial shear and to dilate at larger strains, so developing positive, then negative pore pressures in undrained loading. A soil with an initial void ratio below the ultimate state line (e.g. point "SH" on Figure 12.9) will

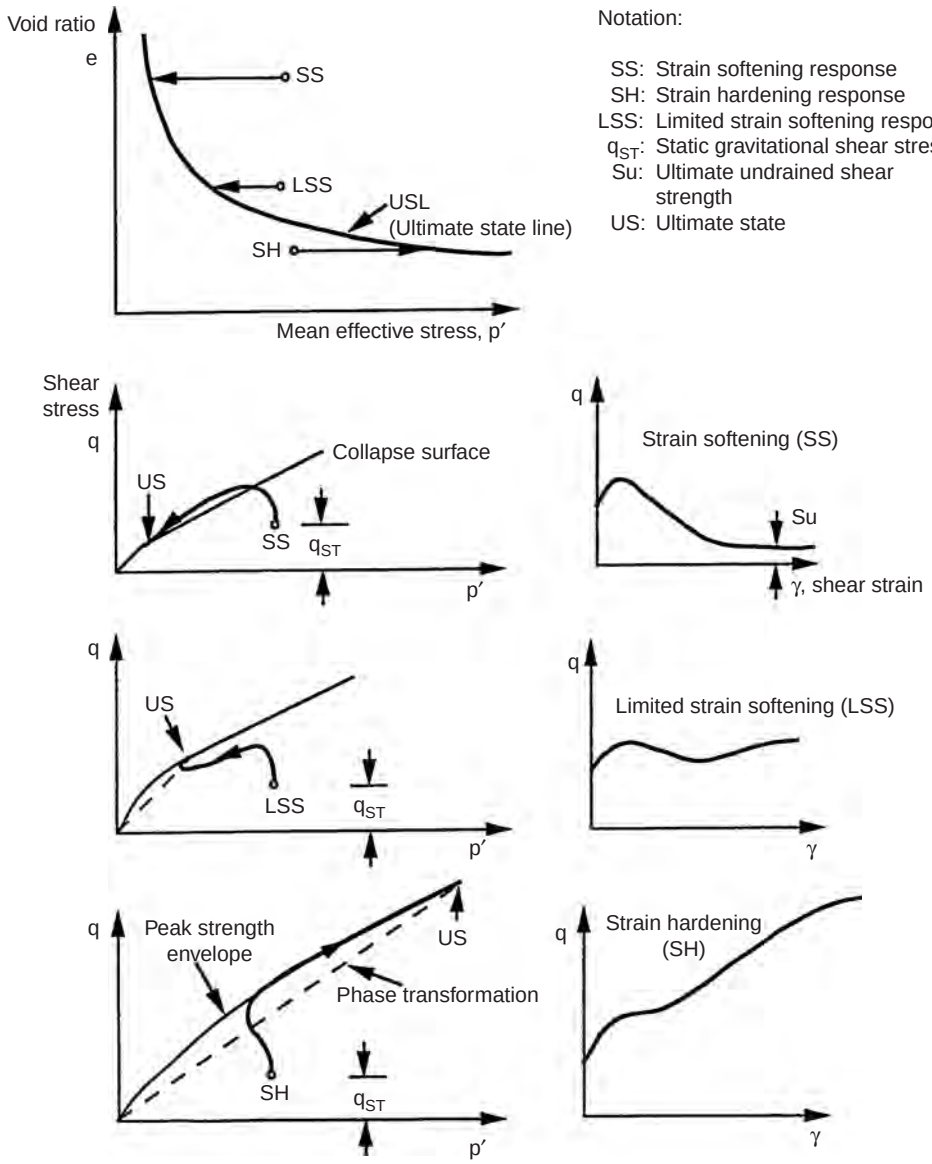


Figure 12.9 Schematic of undrained monotonic behaviour of sand in triaxial compression (Robertson 1994).

tend to dilate, or develop negative pore pressures in undrained loading, giving a strain "hardening" response.

The susceptibility of a soil to static liquefaction is dependent on the particle size distribution, void ratio (or density index), the initial stress conditions and the stress path of loading, e.g. triaxial compression or extension.

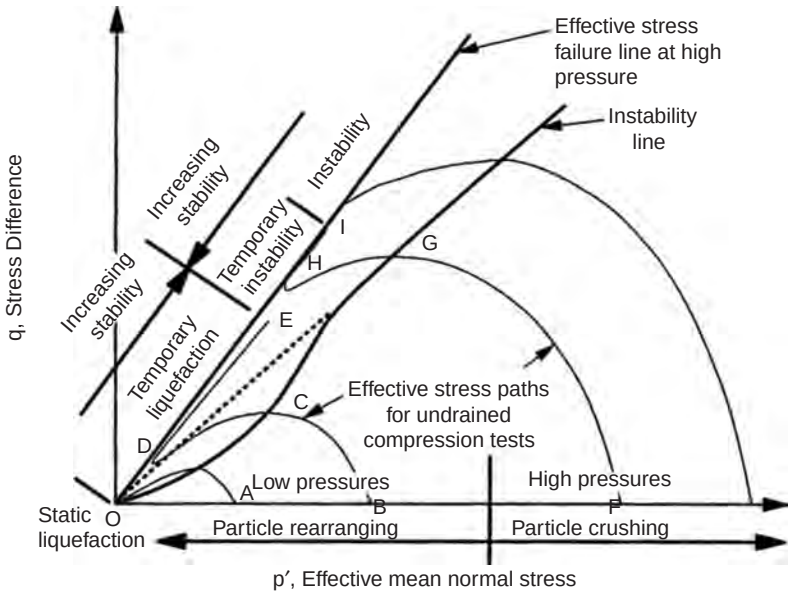


Figure 12.10 Undrained effective stress behaviour of loose silty sands (Lade and Yamamuro, 1997).

Lade and Yamamuro (1997), and Yamamuro and Lade (1997), show that there are four different types of undrained stress paths after the instability line (also known as collapse surface). These are shown in Figure 12.10:

- *Static liquefaction* occurs at low stresses and is characterised by large pore pressure development, resulting in zero effective stresses at low axial strain levels (stress path AO). Increasing confining pressures result in increasing effective stress friction angles in this stress region.
- *Temporary liquefaction* occurs at higher stresses than the static liquefaction region and is characterized by stress path BCDE. Hence the behaviour is contractant from C to D, then dilatant from D to E which occurs at large axial strains. The tendency to dilate (and less likely to contract) increases as the initial confining stresses increase.
- *Temporary instability* (stress path FGHI) is similar to temporary liquefaction except that the amount the stress difference increases beyond the initial peak (i.e. I vs G), is not as large as that exhibited by temporary liquefaction.
- *Instability* – at higher stresses, is due partly to particle crushing.

It should be noted that a soil may reach the instability line (collapse surface) at stress conditions less than the effective stress failure line. Also a soil can be in a stress condition just to the right (stable) of the instability line and cross over the line by change in static stress conditions, e.g. a rise in pore pressures or strains induced for example by an earthquake.

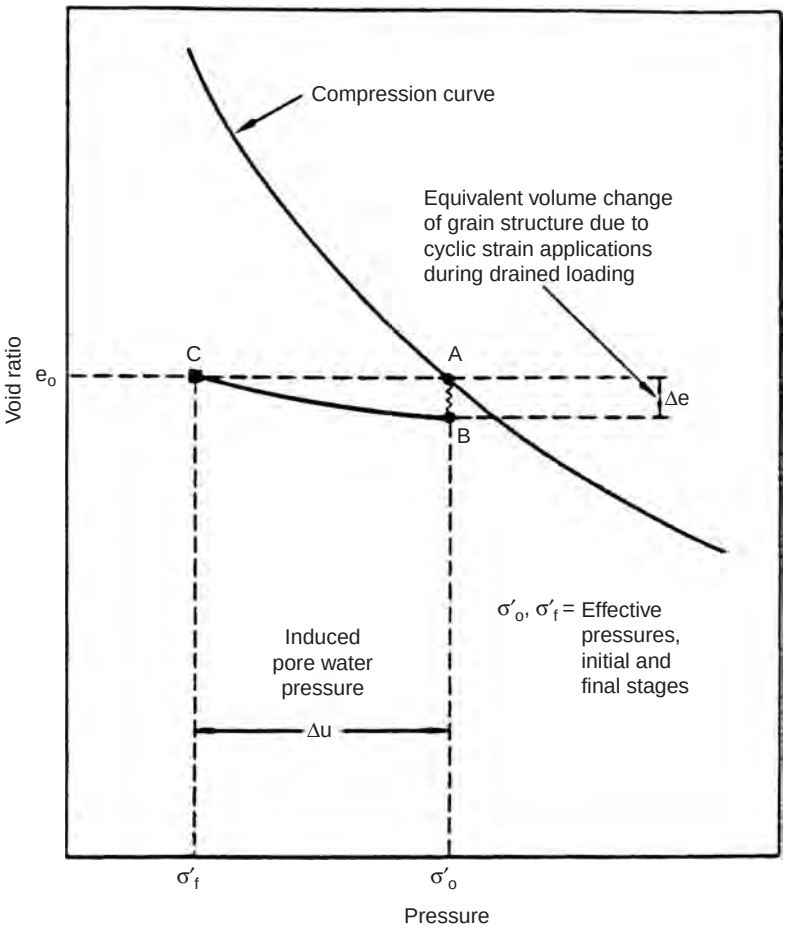


Figure 12.11 Schematic illustration of mechanism of pore pressure generation during cyclic loading (Seed and Idriss 1982; USNRC, 1985, reproduced with permission of ASCE).

12.5.1.2.2 *The effects of cyclic loading*

Cyclic loading, such as that from an earthquake, causes densification of dry granular soils by particle rearrangement due to the back and forth straining. If, however, the soil is saturated and not allowed to drain during cyclic loading, just as in monotonic loading the decrease in volume cannot occur and the tendency to decrease volume is counteracted by an increase in pore pressure and decrease in effective stress – see Figure 12.11.

Hence, soil starting at A and subject to cyclic loading, which would otherwise have ended at B, will in fact, have stresses represented by C where total stress σ_o is taken by σ'_f and Δ_u. The pore pressures must increase to maintain equilibrium in this undrained condition.

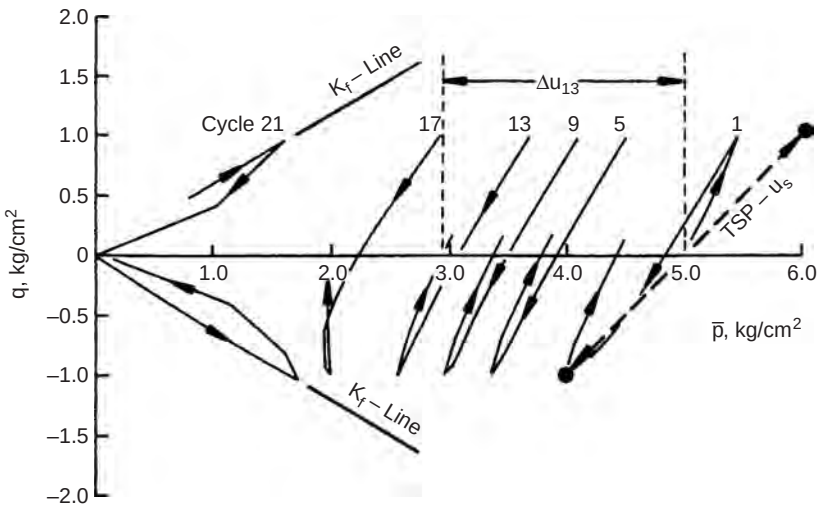


Figure 12.12 Example of effective and total stress paths leading to cyclic liquefaction for isotropically consolidated fine sand in a cyclic triaxial test (Hedberg, 1977; USNRC, 1985).

The pore pressures build up gradually with the number of cycles of loading and only if the pore pressures build up to equal the total stress, does the "initial liquefaction" (effective stress $\sigma' = 0$) condition occur.

The stress path followed by the soil affects the liquefaction potential. Figure 12.12 shows the p′-q stress paths during a cyclic triaxial test, the effective stress path moving from cycle 1 to the left before reaching the failure envelope on the k_f line in cycle 21.

Once the failure line is reached, pore pressures and strain development accelerates. After $\sigma' = 0$ is reached the stress paths are up and down the failure lines passing through or near the origin twice in each cycle as shown. This is the cyclic liquefaction condition.

If the sample is subjected to anisotropic consolidation, or to a constant shear stress over and above the cycled stress, the stress paths are altered significantly, as shown in Figure 12.13.

Here the cycling takes the sample from the failure line (compression side) away to a non-failure condition so, while the soil will continue to strain, the continuous $\sigma' = 0$ condition is not reached. This would be a cyclic mobility condition.

Laboratory tests show that the number of cycles to cause initial liquefaction is dependent on the cyclic stress ratio τ/σ'_o, the relative density (or void ratio related to the ultimate state void ratio), soil particle size and fabric, the stress conditions, and stress path. Figure 12.14 shows some tests on sand showing the effect of relative density and cyclic stress ratio.

It will be noted that for a given cyclic stress ratio, soils at lower relative density require fewer cycles of loading to achieve initial liquefaction.

Table 12.4 shows that the number of cycles of ground motion are related to earthquake magnitude, which can be related qualitatively to the effects shown in

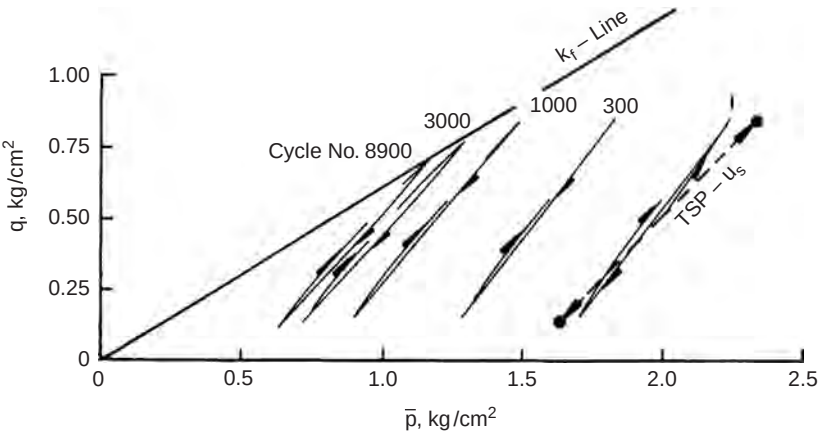


Figure 12.13 Example of effective and total stress paths leading to cyclic mobility on anisotropically consolidated fine sand during cyclic triaxial compression test (Hedberg, 1977; USNRC, 1985).

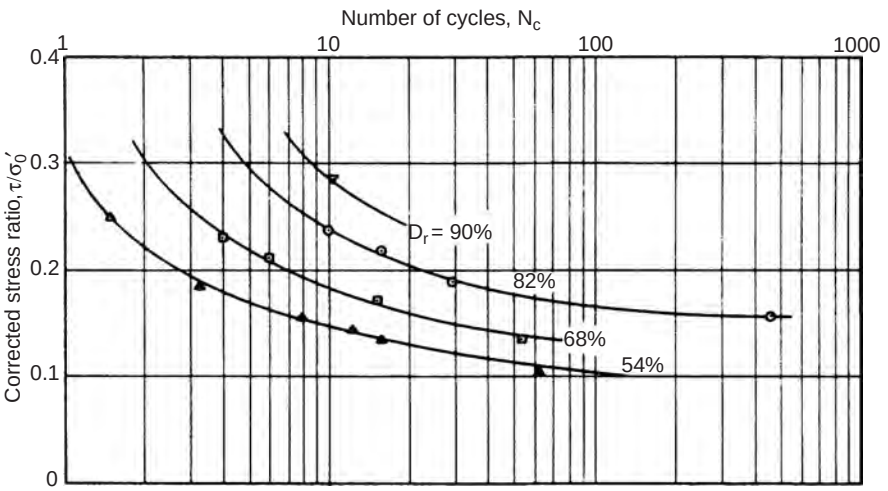


Figure 12.14 Stress ratio τ/σ_o' versus number of cycles to initial liquefaction from tests on a shaking table (De Alba *et al.*, 1976; USNRC, 1985, reproduced with permission of ASCE).

Figure 12.14 to show that only large magnitude earthquakes will have sufficient cycles or large enough cyclic stress ratio to liquefy medium dense to dense soils.

12.5.1.2.3 *Post earthquake behaviour*

As shown in Figure 12.15, the effect of cyclic loading is also to induce strain. If, as shown in Figure 12.15(a), the soil is strain weakening, the strain may be sufficient to

Table 12.4 Representative number of cycles of ground motion versus earthquake magnitude (adapted from Seed and DeAlba, 1986).

Earthquake magnitude	*Number of representative cycles at 0.65 τ_{max}*
8.5	26
7.5	15
6.75	10
6.0	5–6
5.25	2–3

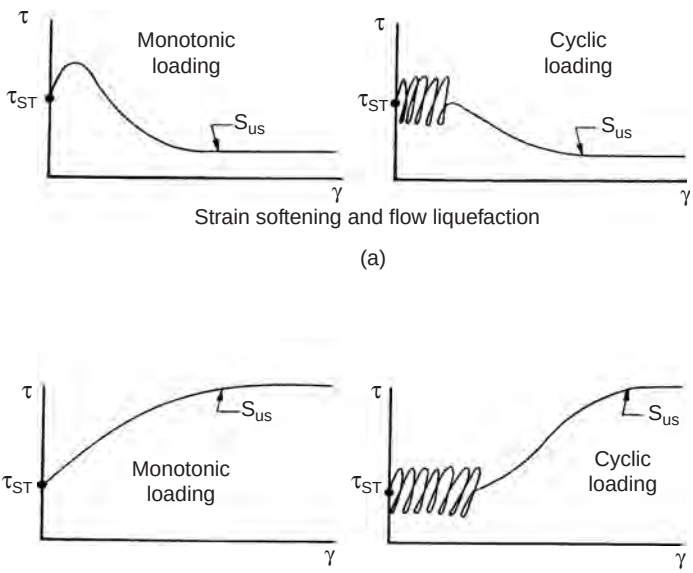


Figure 12.15 Contractive (a) and dilatent (b) soils in undrained static and cyclic loading.

take the soil to the condition that the static gravitational stresses τ_{ST} exceed the available strength, so straining will continue under gravity loads without further cycling by the earthquake. This is the flow liquefaction condition and large, relatively rapid deformations of the slope will occur. For a strain hardening soil, as shown in Figure 12.15(b) the cyclic loading causes deformations during the cycling, but the static

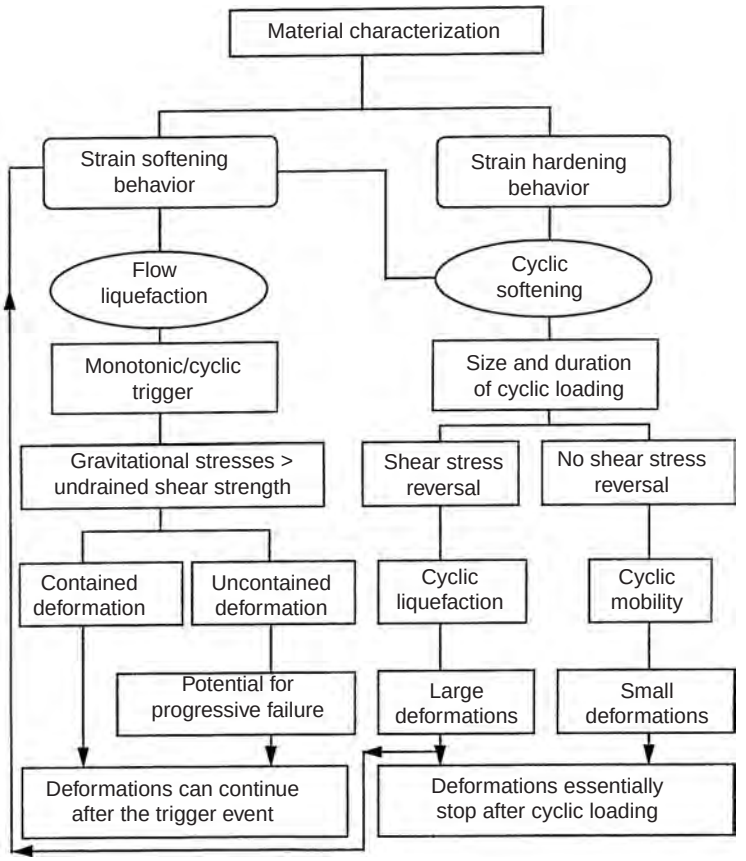


Figure 12.16 Suggested flow chart for evaluation of soil liquefaction (Robertson, 1994; Robertson and Wride, 1997).

gravitational stresses are less than the steady state undrained strength, S_{us}, so the slope will be stable after the cyclic loading ceases.

For slopes partly of strain softening and partly of strain hardening soil overall instability will occur after the cyclic stresses cease only if after stress redistribution in the strain softening soil, the remaining soil cannot support the gravitational shear stress. Robertson and Wride (1997) suggest a flow slide can only occur if a kinematically admissible mechanism can develop. Whether a flow slide would occur would also depend on the factor of safety of the slope using S_{us} in the liquefied soil. If it is significantly less than 1.0, large, rapid deformations will occur based simply on the momentum effects.

12.5.1.3 *Suggested flow chart for evaluation of soil liquefaction*

Robertson (1994) and Robertson and Wride (1997) suggest that the flow chart shown in Figure 12.16 be used for the evaluation of soil liquefaction. The following summary

of the conditions for flow liquefaction, cyclic liquefaction and cyclic mobility is taken from those papers:

(a) *Flow liquefaction*
- Applies to strain softening soils only.
- Requires a strain softening response in undrained loading resulting in constant shear stress and effective stress.
- Requires *in situ* shear stresses greater than the ultimate or minimum undrained shear strength, as illustrated in Figure 12.15.
- Either monotonic or cyclic loading can trigger flow liquefaction.
- For failure of a soil structure to occur, such as a slope, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics and ground geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism occurs.
- Can occur in any meta-stable saturated soil, such as very loose granular deposits, very sensitive clays, and loess (silt) deposits.

(b) *Cyclic liquefaction*
- Requires undrained cyclic loading during which shear stress reversal occurs or zero shear stress can develop (i.e. occurs when *in situ* static shear stresses are low compared to cyclic shear stresses).
- Requires sufficient undrained cyclic loading to allow effective stresses to reach essentially zero.
- At the point of zero effective stress no shear stress exists. When shear stress is applied, pore pressure drops as the material tends to dilate, but a very soft initial stress strain response can develop, resulting in large deformations.
- Deformations during cyclic loading can accumulate to large values, but generally stabilize when cyclic loading stops. The resulting movements are due to external causes and occur only during the cyclic loading.
- Can occur in almost all saturated sands provided that the cyclic loading is sufficiently large in magnitude and duration.
- Clayey soils can experience cyclic liquefaction but deformations are generally small due to the cohesive strength at zero effective stress. Rate effects (creep) often control deformations in cohesive soils.

(c) *Cyclic mobility*
- Requires undrained cyclic loading during which shear stresses are always greater than zero, i.e. no shear stress reversal develops.
- Zero effective stress will not develop.
- Deformations during cyclic loading will stabilize, unless the soil is very loose and flow liquefaction is triggered. The resulting movements are due to external causes and occur only during the cyclic loading.
- Can occur in almost any saturated sand provided that the cyclic loading is sufficiently large in magnitude and duration, but no shear stress reversal occurs.
- Cohesive soils can experience cyclic mobility, but rate effects (creep) usually control deformations.

Note that strain softening soils also experience cyclic softening (cyclic liquefaction or cyclic mobility) depending on the ground geometry.

In Figure 12.16 the first step is to evaluate the material characteristics in terms of a strain softening or strain hardening response. If the soil is strain softening, flow liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the ultimate or minimum strength. The trigger mechanism can be either monotonic or cyclic. Whether a slope or soil structure will fail and slide will depend on the amount of strain softening soil relative to strain hardening soil within the structure, the brittleness of the strain softening soil and the geometry of the ground. The resulting deformations of a soil structure with both strain softening and strain hardening soils will depend on many factors, such as distribution of soils, ground geometry, amount and type of trigger mechanism, brittleness of the strain softening soil and drainage conditions.

If the soil is strain hardening, flow liquefaction will generally not occur. However, cyclic softening can occur due to cyclic undrained loading, such as earthquake loading. The amount and extent of deformations during cyclic loading will depend on the density of the soil, the magnitude and duration of the cyclic loading and the extent to which shear stress reversal occurs. If extensive shear stress reversal occurs, it is possible for the effective stresses to reach zero and hence, cyclic liquefaction can take place. When the condition of essentially zero effective stress is achieved, large deformations can result. If cyclic loading continues, deformations can progressively increase. If shear stress reversal does not take place, it is generally not possible to reach the condition of zero effective stress and deformations will be smaller, i.e. cyclic mobility will occur.

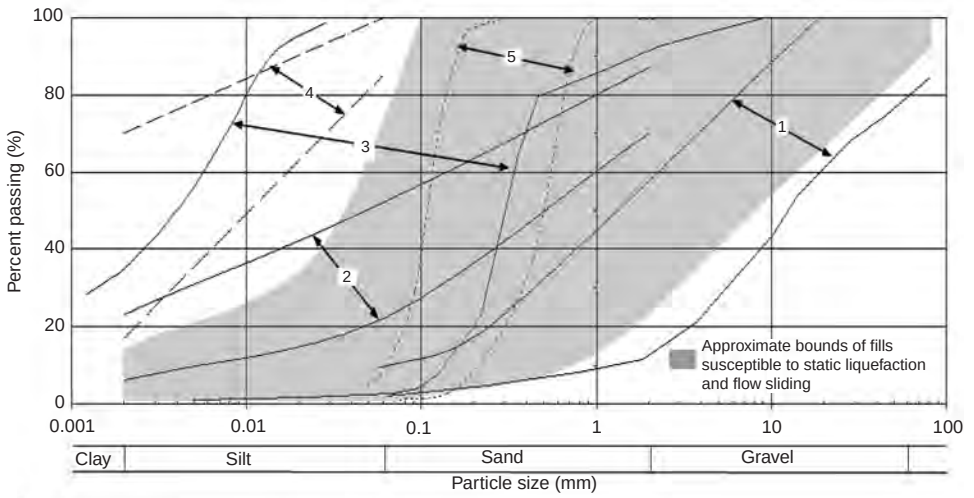
Both flow liquefaction and cyclic liquefaction can cause very large deformations. Hence it can be very difficult to clearly identify the correct phenomenon based on observed deformations following earthquake loading. Earthquake-induced flow liquefaction movements tend to occur after the cyclic loading ceases, due to the progressive nature of the load redistribution. However, if the soil is sufficiently loose and the static shear stresses are sufficiently large, the earthquake loading may trigger essentially "flow liquefaction" within the first few cycles of loading. Also, if the soil is sufficiently loose, the ultimate undrained strength may be close to zero with an associated effective confining stress very close to zero (Ishihara, 1993). Cyclic liquefaction movements, on the other hand, tend to occur during the cyclic loading since it is the inertial forces that drive the phenomenon.

12.5.2 Soils susceptible to liquefaction

12.5.2.1 *Methods based on soil classification and in situ moisture content*

It has long been recognized that saturated sands, silty sands and gravelly sands are susceptible to liquefaction. As described by Seed *et al.* (2003) as more data has been gathered it has been recognized that coarser gravelly soils and silty and silty clay soils may also be subject to liquefaction.

Hunter and Fell (2003a, b) gathered data from case studies where flow liquefaction had occurred, mainly under static loading conditions. These would also be reasonably



1 - Coarse grained coal mine waste (Dawson *et al.*, 1998; Taylor, 1984; Bishop *et al.*, 1969; Hutchinson, 1989)
2 - Loose silty sand fills, Hong Kong (upper and lower quartile of pre 1977 fills (HKIE 1998))
3 - Hydraulically placed mine tailings and fills in dam embankments (various published sources)
4 - Sensitive clays (indicative limits from: Lefebvre, 1996; Bentley & Smalley, 1984; Mitchell & Markell, 1974; Hutchinson, 1961, 1965)
5 - Sub-aqueous slopes, natural and fill slopes (Koppejan *et al.*, 1948; Kramer, 1988; Sladen & Hewitt, 1989; Cornforth *et al.*, 1974)

Figure 12.17 Particle size gradation of soils susceptible to flow liquefaction under static and earthquake loading (Hunter and Fell, 2003a, b).

applicable to earthquake loading. Figure 12.17 shows the data separated into classes of slope, with a suggested range of soils susceptible to flow liquefaction.

Seed *et al.* (2003) note however that gravelly soils are often more permeable so can dissipate cyclic generated pore pressures unless they are contained within finer strata which inhibit drainage during cyclic loading.

It should be recognized that particle size distribution alone is not able to identify soils which are susceptible to liquefaction.

There are a number of well established methods available to identify soils which are susceptible to liquefaction based on the particle size gradation, plasticity and *in situ* moisture content of the soil. These include:

(i) "Chinese Criteria" (Seed and Idriss, 1982)

Seed *et al.* (2003) and Boulanger and Idriss (2007), Idriss and Boulanger (2008) all indicate that the Chinese Criteria (Seed and Idriss, 1982) criteria are outdated and should not be used.

(ii) Seed et al. (2003) criteria

Seed *et al.* (2003) gathered a significantly larger database of field performance than available to Seed and Idriss (1982) and also used data from laboratory tests. Based on these they found that the Seed and Idriss (1982) method was non-conservative and soils with higher clay size fraction than suggested by that method had liquefied. They also

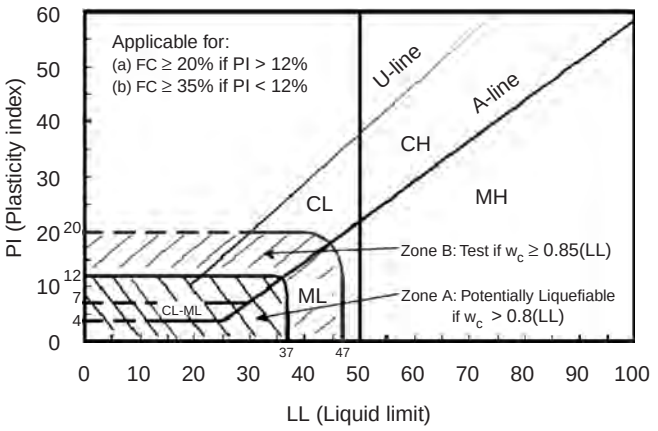


Figure 12.18 Seed *et al.* (2003) method for assessing liquefiable soil types (Seed *et al.*, 2003).

found that soil plasticity was more important than clay size fraction. Based on this they suggested that Figure 12.18 be used. Seed (2010) presented additional information in support of this conclusion.

Seed *et al.* (2003) indicate that for soils with sufficient fines content that the fines separate the coarser particles and control behaviour:

- Soils within Zone A represents soils which are potentially susceptible to "classic" cyclic liquefaction.
- Soils within Zone B may be liquefiable.
- Soils within Zone C (soils not plotting in Zones A or B), are not generally susceptible to "classic" cyclic liquefaction but should be checked for potential sensitivity (loss of strength with remoulding or monotonic accumulation of shear deformation).

They define "classic" cyclic liquefaction as referring to soils which experience significant loss of strength and stiffness due to cyclic pore pressure generation.

(iii) Robertson and Wride (1998) soil behaviour type index Ic for Cone Penetration Tests

This method described in Youd *et al.* (2001; from Robertson and Wride (1998) calculates the soil behaviour type index I_c as follows:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F^2)]^{0.5} \tag{12.10}$$

where

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma'_{vo}]^n \tag{12.11}$$

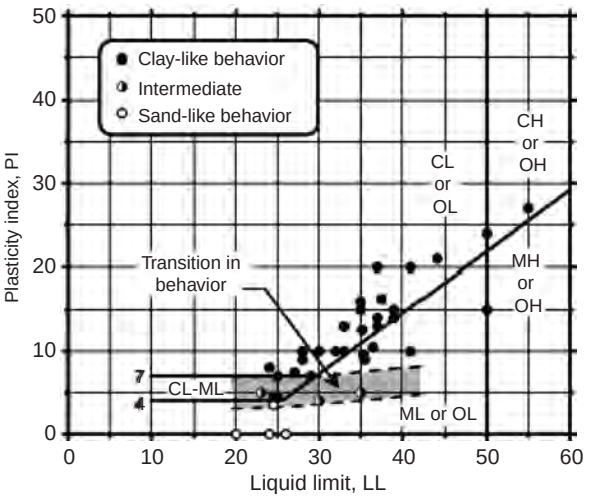


Figure 12.19 Atterberg limits chart showing representative values for soils which exhibited clay-like; sand-like and intermediate behaviour in cyclic loading (Boulanger and Idriss, 2006 with permission from ASCE).

and the Normalized Friction Ratio F from

$$F = [fs/(q_c - \sigma_{vo})]100\% \quad (12.12)$$

where fs = CPT sleeve resistance, (kPa); σ_{vo} = total vertical stress at test level, (kPa) $\sigma_{vo'}$ = effective vertical stress at test level; q_c is the cone resistance and assume n = 1.0 for the initial calculation.

If $I_c > 2.6$, the soil is classified as clayey, so is unlikely to liquefy.

If $I_c < 2.6$, the soil is likely to be granular, so recalculate Q, and I_c using n = 0.5. If the recalculated $I_c < 2.6$, the soil is non-plastic and granular.

If the recalculated $I_c > 2.6$, the soil is likely to be silty, possible plastic, so I_c should be recalculated using n = 0.7 and this value of I_c used to calculate liquefaction resistance as described in Section 12.5.3.6. Such soils are marginal and Youd *et al.* (2001) suggest they be sampled and tested to verify behaviour and/or checked using other criteria.

The first author has been involved in a project where soils with $I_c > 2.6$ but <2.9 were considered as potentially liquefiable by the consultant and peer reviewer, both of whom were experts in the field. This approach was also independently taken by the authors on another project where the other criteria indicated the soils might be liquefiable.

(iv) Boulanger and Idriss (2006)

Boulanger and Idriss (2006) and Idriss and Boulanger (2008) present Figures 12.19 and 12.20 to distinguish between soils which exhibit sand-like behaviour and liquefy, and soils which exhibit clay-like behaviour and cyclic softening. This is based on cyclic laboratory tests. There is an intermediate zone which is not clearly defined but

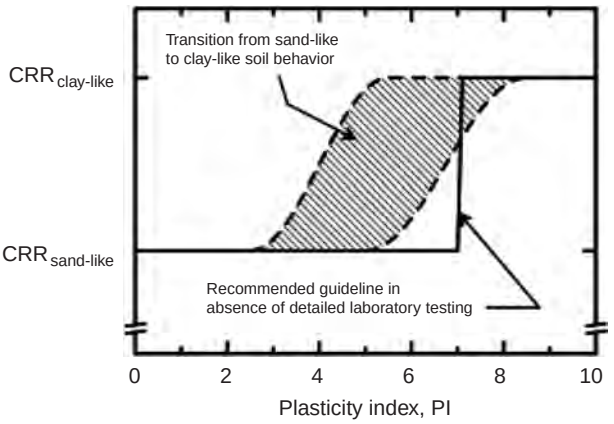


Figure 12.20 Schematic illustration of the transition from sand-like to clay-like behaviour with increasing plasticity index and the recommended guideline for practice. (Boulanger and Idriss, 2006 with permission from ASCE).

they conclude that soils with a PI ≥ 7 can reasonably be expected to exhibit clay-like behaviour. Soils with PI < 7 should be considered soils that probably exhibit sand-like behaviour (that is they are liquefiable) unless shown to be otherwise through detailed *in situ* or laboratory testing. They indicate that these criteria are not meant to be equivalent to those listed above but are intended to provide guidance in choosing engineering procedures.

(v) Robertson (2010) for Cone Penetration Tests

Robertson (2010) presents Figure 12.21 to be used to identify soils susceptible to flow liquefaction and cyclic softening. These are based on case data and theory. The boundary between Zones A1, A2 and B, C can be approximated by $I_c = 2.6$ as above, and the boundary between Zones A1, B and A2, C by $Q_{1n} = 70$. (Note that Q_{1n} and Q in (iii) are both normalized cone resistance). The data on which this is based are shown in Figure 12.22.

12.5.2.2 *Discussion and recommended approach*

Seed (2010) discusses the Boulanger and Idriss (2006) method and plots field case data which he indicates shows the Boulanger and Idriss (2006) criteria are non-conservative, with case data showing liquefaction of soils occurring with PI > 7 contrary to the Boulanger and Idriss (2006) criteria. Given these data it would appear that caution should be applied in relying on the Boulanger and Idriss (2006) criteria alone to identify soils which may liquefy.

The authors noted that for a dam where foundation liquefaction was thoroughly assessed using SPT and CPT data the Robertson and Wride (1998) method identified a number of strata as non liquefiable where the Seed and Idriss (1982) and Seed *et al.* (2003) methods both indicated the soil was potentially liquefiable. In view of this it seems unwise to rely upon CPT data alone.

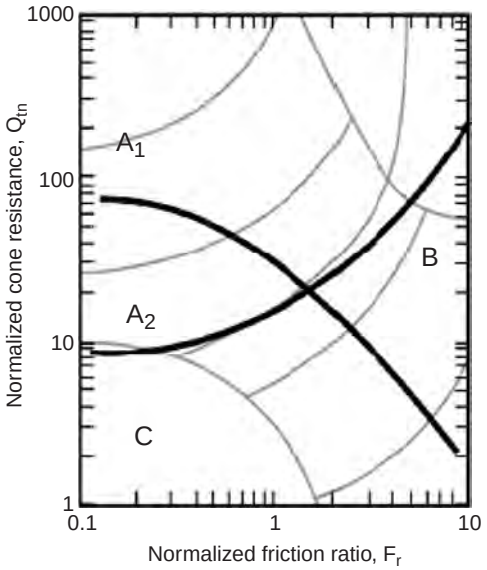


Figure 12.21 Cone Penetration Test/Soil Behaviour Type chart for liquefaction and cyclic softening. (Robertson, 2010 with permission from ASCE). *Cohesionless soils*, A1, cyclic liquefaction possible; A2 cyclic liquefaction and flow liquefaction possible; *Cohesive soils*, B cyclic softening possible; C cyclic softening and flow liquefaction possible depending on soil sensitivity.

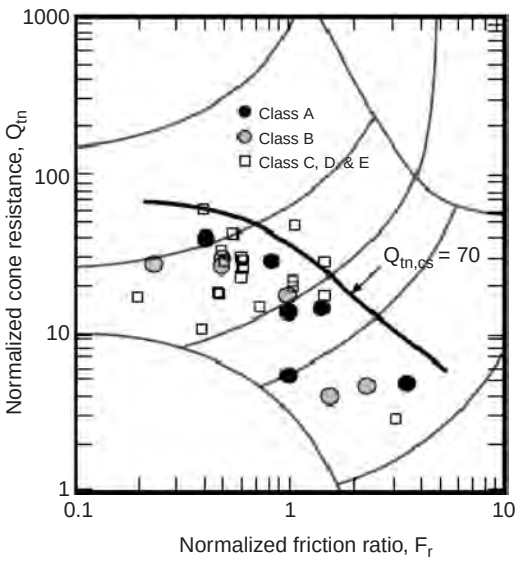


Figure 12.22 Mean values of normalised CPT data from flow liquefaction case histories (Robertson, 2010 with permission from ASCE).

Table 12.5 Assessing the likelihood that a soil is potentially liquefiable.

Result of assessment	*Likelihood this strata is potentially liquefiable*
Three methods indicate liquefiable	Highly likely to certain
Two methods indicate liquefiable	Likely
Only Seed *et al.* (2003) indicates liquefiable, others do not	Likely to Possible Depending if Zone A or Zone B
Only Robertson and Wride (1998) indicates liquefiable, others do not.	Unlikely
Three methods indicate non liquefiable	Liquefaction can be discounted

It is suggested that:

(1) Use the methods of Seed *et al.* (2003) and Boulanger and Idriss (2006) to assess whether a soil is potentially liquefiable. The methods should be used with all required data inputs including *in situ* moisture content, Atterberg limits, % fines.
(2) Where Cone Penetration Test data is also available (which is preferable) also use the method of Robertson and Wride (1998) or Robertson (2010).
(3) Use these data together to assess the likelihood the soils are potentially liquefiable.
(4) Consider the results as shown in Table 12.5.

Where soils plot in Zone B of the Seed *et al.* (2003) method they should be considered as potentially susceptible to liquefaction and /or cyclic softening. If their behaviour becomes critical to the assessment of the safety of the dam either the soil should be assumed to be liquefiable or undisturbed samples should be subject to cyclic loading tests rather than relying on semi-empirical methods to predict behaviour.

12.5.2.3 *Methods based on geology and age of the deposit*

Idriss and Boulanger (2008) refer to Youd and Perkins (1978) and indicate that the soils which are most susceptible to liquefaction are non plastic or very low plasticity fill, alluvial, fluvial, marine and deltaic soils. They indicate soils are most susceptible when they are recently deposited, becoming more resistant to liquefaction as they become older. The most susceptible soils are Holocene or younger soils, Pleistocene soils are low or very low susceptibility except for loess.

Youd *et al.* (2001) discuss the effects of the age of the deposit and conclude that for man-made structures such as fills and embankment dams, aging effects are minimal and corrections for age should not be applied in calculating liquefaction resistance.

Andrus *et al.* (2009) and Hayati and Andrus (2009) present methods for correcting for the age of the soils being assessed for liquefaction. These are discussed further in Section 12.5.3.9. These include data showing liquefaction has occurred in Pleistocene sands.

It is concluded that geological age alone cannot be used as a means of screening for liquefaction susceptibility. It may be taken as additional information to be considered in assessing the likelihood of liquefaction.

12.5.3 The "simplified procedure" for assessing liquefaction resistance of a soil

12.5.3.1 *Background to the simplified method*

The most widely accepted, simplest and most practical method of assessing whether there is a potential for liquefaction for horizontal ground conditions was developed originally by Professor H.B. Seed and his co-workers (Seed and Idriss, 1971; Seed, 1979b; Seed and Idriss, 1982; Seed *et al*., 1985b; Seed and De Alba, 1986).

The method is semi-empirical and is based on the maximum acceleration induced by the earthquake a_{max}, the SPT "N" value corrected for the SPT hammer energy and for overburden pressure $(N_1)_{60}$, earthquake magnitude (M_W) and fines content of the soil (% passing 0.075 mm). It is based on recorded cases of liquefaction during earthquakes in USA, Japan and China.

There has been a gradual development of refinements of the method, including greater application of the cone penetration test, modification to earthquake magnitude corrections and to the corrections for high overburden stress, static shear stresses and age of the soil deposit.

In 1996 a NCEER workshop on the evaluation of Liquefaction Resistance of Soils was attended by 21 experts, who reached a consensus report on the state of the practice at the time. This is reported in NCEER (1997) and Youd *et al*. (2001).

Since then there have been a number of developments:

(i) Seed *et al*. (2003) prepared a University of California at Berkeley Report, which presented updated methods based on enlarged databases of cases, using a probabilistic approach.
(ii) Idriss and Boulanger (2008) from University of California at Davis produced Monograph MNO-12 for the Earthquake Engineering Research Institute (EERI). This reviewed the available literature and updated many aspects of the method.
(iii) Seed (2010) issued a University of California at Berkeley Geotechnical Report, which was critical of a number of aspects of Idriss and Boulanger (2008).
(iv) Idriss and Boulanger (2010) responded to this criticism and provided more details on the basis of their EERI Monograph.

As a result of the differences of opinion EERI set up an ad hoc committee to review what they termed the "*strong differences of opinion, often personalised and polarized*". This is reported in Finn *et al*. (2010). Further comments on this review were given in:

(v) Youd (2010) provided some personal insights to the discussion.
(vi) Cox and Griffiths (2011) contributed by comparing the use of the Youd *et al*. (2001), Seed *et al*. (2003) and Idriss and Boulanger (2008) approaches on three sites.

As a result of these developments it has been necessary in preparing this book to review these documents and provide guidance on which method or methods should be used and details which need to be considered.

12.5.3.2 *Discussion of differences between the Youd et al. (2001), Seed et al. (2003) and Idriss and Boulanger (2008) methods*

Cox and Griffiths (2011) used the three SPT based methods of Youd *et al.* (2001), Seed *et al.*, (2003) and Idriss and Boulanger (2008) to analyse three case studies. They found that they did not give greatly differing factors of safety against liquefaction in the upper 24 m of the soil profiles. They found that the Idriss and Boulanger (2008) method typically yields the highest factor of safety at depth, generally ranging from 5–15% higher than the others, but the differences were much greater at raw blow counts greater than 23. This was attributed to Idriss and Boulanger (2008) using higher stress correction K_σ values but more importantly higher overburden stress correction C_N values. These combined with the shape of the CRR curve at high $(N_1)_{60CS}$ to give the larger differences.

They found that in the upper soil profile the individual inputs to the methods varied quite a lot but the effects were often balanced by other factors. In particular they found that quite different stress reduction coefficient r_d values did not result in greatly different factors of safety (cyclic resistance ratio/cyclic stress ratio; CRR/CSR. This is consistent with the advice from Idriss and Boulanger (2008) that because the case histories used in the development of their method were analysed using their methods for accounting for depth effects, earthquake magnitude and fines content, it is important that when using their method the same methods as they used for these effects are adopted.

It is apparent that approach would also apply if using the Seed *et al.* (2003) method. In particular values of r_d, C_N, and K_σ should not be substituted from one method to another.

Overall it is concluded that:

(a) All three SPT based methods may be used and be expected to result in similar factors of safety against liquefaction in the upper 15 metres.

(b) The SPT based method as described in Idriss and Boulanger (2008) and Boulanger and Idriss (2012) incorporates many refinements developed since the Youd *et al.* (2001) method was published and its use is preferred. However where K_σ values are critical to the outcome both the Idriss and Boulanger (2008) and Youd *et al.* (2001)/Hynes and Olsen (1999) values should be used and where the Idriss and Boulanger (2008) values are controlling the outcome of the assessment of the likelihood of liquefaction expert advice should be sought.

(c) As can be seen in Figures 12.23 and 12.24 there is not a large difference between the NCEER/NSF Workshops (Youd *et al.* (2001) and Idriss and Boulanger (2004, 2008) plots. Within the context of the uncertainty in ground motions the differences are small and will make little difference in a risk based analysis. The Cetin *et al.* (2004) (Seed *et al.*, 2003) curve plots to the right of the others. This apparent significant difference between the three authors can partly be explained by the way in which the database has been analysed by Cetin *et al.* (2004). However Idriss and Boulanger (2010) and Boulanger and Idriss, (2012) present a discussion of the data points which control the position of the curve. They cast doubt on the validity of 8 key points on the grounds of incorrect assignation of liquefaction/no liquefaction of four points and significant numerical errors between

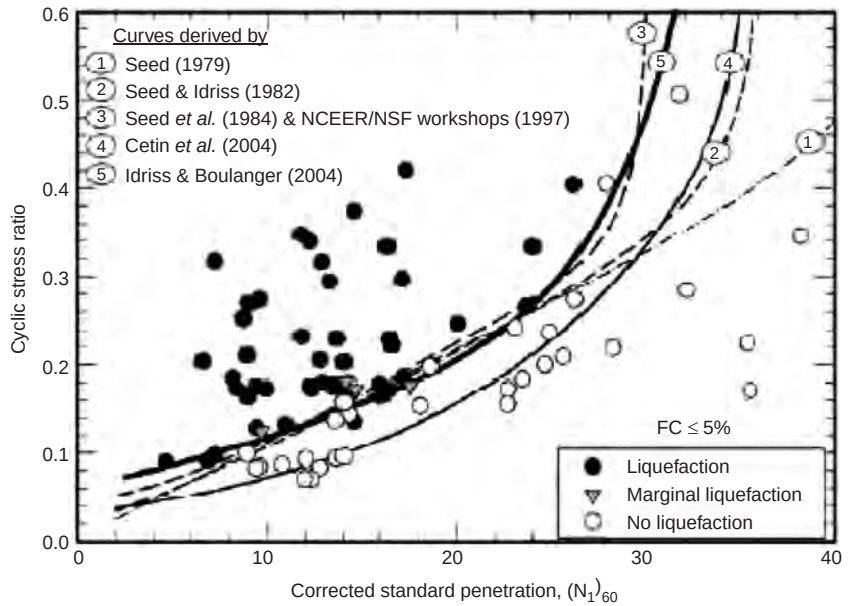


Figure 12.23 Curves relating cyclic resistance ratio (CRR) to $(N_1)_{60CS}$ values for earthquake magnitude Mw = 7.5 and an effective overburden stress of 100 kPa (Idriss and Boulanger, 2008).

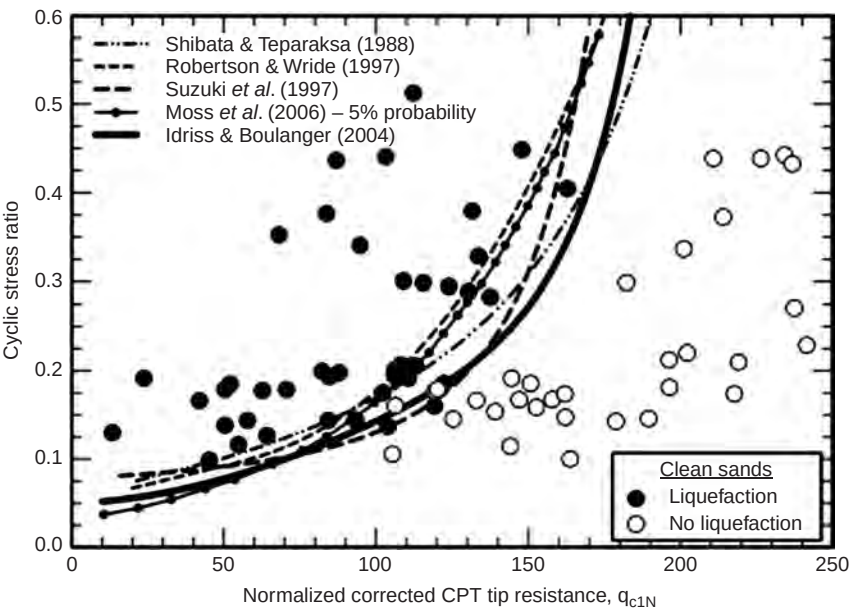


Figure 12.24 Curves relating cyclic resistance ratio (CRR) to $(q_{c1N})_{cs}$ values for earthquake magnitude Mw = 7.5 (Idriss and Boulanger, 2008).

the r_d values used to develop their correlations and the r_d values computed using their applicable equation for four others.

(d) There is less information available to make an assessment of CPT based methods. However on what is presented all three CPT based methods may be used for assessing factors of safety against liquefaction in the upper 15 or 20 metres.

(e) It is apparent there is considerable uncertainty in the estimation of static shear stress correction factor K_α. It is suggested that Idriss and Boulanger (2008) be used but marginal cases should be assumed liquefiable, or laboratory tests and analysis of stresses should be carried out to assess the effects.

(f) The simplified methods are not considered reliable below about 15 or 20 metres. Most of the uncertainty discussed above is below that depth. For important decisions relating to marginally liquefiable soils below that depth expert advice should be obtained.

(g) There is considerable uncertainty in many of the parameters used in these methods. Users should not regard the outcomes as clearly liquefiable or not liquefiable for soils plotting near the boundaries. This can be readily accounted for in a probabilistic framework.

(h) The effects of ageing may be considered in Pleistocene and Tertiary age soils but not too much reliance should be made on these effects. The effects of ageing should be considered for recently deposited mine tailings and dredged fills.

The following Sections detail the simplified method based mainly on the Idriss and Boulanger (2008) approach.

Those carrying out liquefaction assessments should refer to the original references regardless of which method is being used.

It has been proposed by Finn *et al.* (2010) that a Workshop similar to the 1996 Workshop be convened. Practitioners should seek that publication if and when it is produced. In the meantime they should keep up to date with the literature particularly that in high quality refereed journals.

12.5.3.3 *The simplified method – outline*

The method requires the calculation or estimation of two variables for evaluation of liquefaction resistance of soils:

(a) The Cyclic Stress Ratio, CSR, which is a measure of the cyclic load applied to the soil by the earthquake.

(b) The Cycle Resistance Ratio, CRR, which is the capacity of the soil to resist liquefaction.

The CRR is estimated from Standard Penetration Tests (SPT), Cone Penetration Tests (CPT) or, less frequently, the shear wave velocity. Figure 12.25 shows the CRR based on SPT as proposed by Idriss and Boulanger (2008) and Boulanger and Idriss (2012).

If the CSR is greater than the CRR, liquefaction is likely to occur.

For most purposes a simple comparison of CSR with CRR with engineering judgement will be sufficient. However the methods available allow for the uncertainty in the boundary between liquefiable and non-liquefiable soils. Idriss and Boulanger (2008),

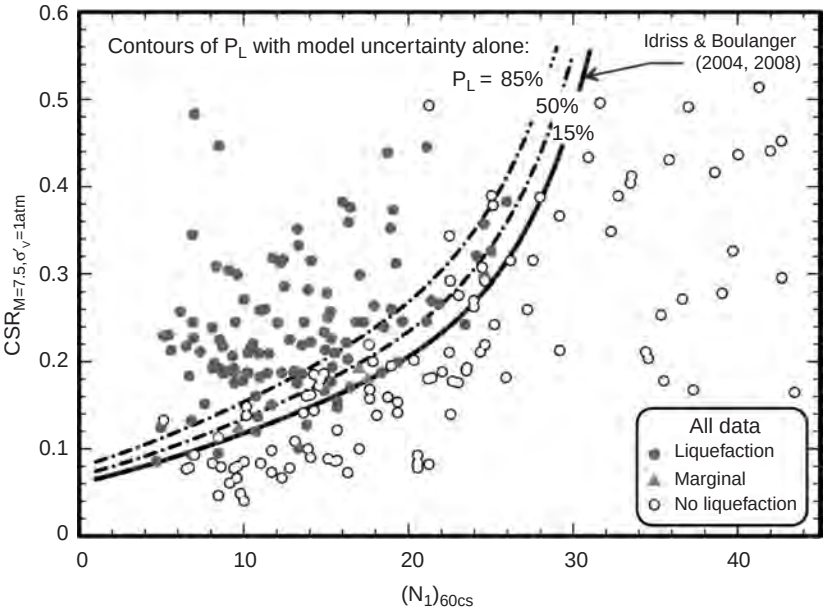


Figure 12.25 SPT Clean-sand base Cyclic Resistance Ratio curves for magnitude 7.5 earthquakes and an effective overburden stress of 100 kPa, showing probabilities of liquefaction of 15, 50 and 85% (Boulanger and Idriss, 2012 with permission from ASCE).

Boulanger and Idriss (2012) indicate that the base curve in Figure 12.25 represents a probability of liquefaction of 16%. That is, to have a CRR greater than CSR is not a guarantee of no liquefaction. These uncertainties are only the model uncertainties and do not include the uncertainty relating to ground conditions.

It is emphasised that the methods need to be used with reasonable engineering judgement and should not be regarded as giving precise outcomes.

The methods are applicable to soils which are judged to be potentially liquefiable as discussed and summarized in Section 12.5.2.1.

12.5.3.4 *Evaluation of Cyclic Stress Ratio (CSR)*

The Cyclic Stress Ratio, CSR, is calculated from:

$$\mathrm{CSR} = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \tag{12.13}$$

where a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively and r_d = stress reduction coefficient, which Idriss and

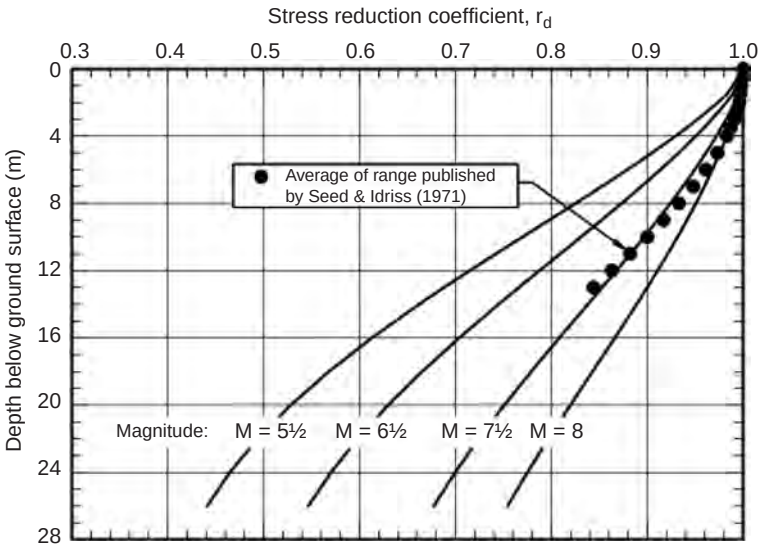


Figure 12.26 Variation of the stress reduction factor r_d with depth and earthquake magnitude (Idriss, 1999).

Boulanger (2008) indicate can be calculated from:

$$r_d = \exp(\alpha(z) + \beta(z)M_W) \tag{12.14}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \tag{12.15}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \tag{12.16}$$

where z = depth below ground surface in metres and the arguments inside the sine terms are in radians.

Figure 12.26 shows plots of the method which is based on Idriss (1999), and the Seed and Idriss (1971) method, which is what was recommended in Youd *et al.* (2001).

Seed *et al.* (2003) use a multiple regression equation for r_d which is a function of depth, M_w, a_{max} and shear wave velocity, Vs, as a measure of stiffness of the strata. They show that the Idriss (1999) method over-estimates r_d when compared to a large number of site response analyses. They show that their method, later published as Cetin *et al.* (2004), fits the data better.

As advised by both sets of authors, users should use the r_d relationship of the author whose method they are using because the analysis of the database to produce the CRR plot uses this relationship.

It should be recognized that there is some uncertainty in r_d. This uncertainty is greater at depth. Idriss and Boulanger (2008) discuss this and recommend that the CSR (or equivalent r_d values) at depths greater than about 20 m should be based on

response studies. These require sufficient subsurface site characterization of the site and should allow for the variability in possible input motions.

If assessments have been made using the Youd *et al.* (2001) method and hence based on Seed and Idriss (1971) they would appear to be somewhat conservative for $M_w < 7.5$ because both Idriss (1999) and Cetin *et al.* (2004) support using lower r_d values.

The peak horizontal acceleration at the ground surface should be calculated allowing for amplification or de-amplification of the peak horizontal bedrock acceleration estimated in the absence of increased pore pressure or the onset of liquefaction.

If there is an existing embankment, the peak horizontal acceleration should be estimated at the crest of the embankment using the same assumptions and from this and the ground surface value, an acceleration representative of what would apply to a critical failure surface estimated. Alternatively, and better, estimate the cyclic stresses directly at the level of the potentially liquefiable layer using dynamic analyses of the embankment.

Youd *et al.* (2001) indicate that the peak horizontal bedrock acceleration is the geometric mean of the two horizontal components (N-S and E-W) where they are available.

To avoid bias in the analysis from short duration, high frequency acceleration estimated from near-field small magnitude earthquakes, the peak horizontal acceleration should be based on the influence of earthquakes Magnitude 5 or greater.

12.5.3.5 *Evaluation of Cyclic Resistance Ratio for M7.5 earthquakes ($CRR_{7.5}$) from the Standard Penetration Tests using the Boulanger and Idriss (2012), Idriss and Boulanger (2008) method*

The CRR for M7.5 earthquakes can be estimated from the curves in Figure 12.25. This plot requires the $(N_1)_{60CS}$ which is the $(N_1)_{60}$ with correction for fines content as detailed below.

Idriss and Boulanger derived an expression for the $CRR_{7.5}$ which fits the 15% probability curve. Some may wish to use this in spreadsheet calculations but if this is done it should be recognized it represents the 15% probability curve and is not the absolute boundary between liquefiable and non-liquefiable soils.

The equation is:

$$CRR_{7.5} = \exp\left(\frac{(N_1)_{60CS}}{14.1} + \left[\frac{(N_1)_{60CS}}{126}\right]^2 - \left[\frac{(N_1)_{60CS}}{23.6}\right]^3 + \left[\frac{(N_1)_{60CS}}{25.4}\right]^4 - 2.8\right) \tag{12.17}$$

The $(N_1)_{60}$ is the SPT blow count normalised to an effective overburden pressure of 100 kPa, hammer energy of 60%, borehole diameter, rod length and sampling method. $(N_1)_{60}$ is given by:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{12.18}$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy

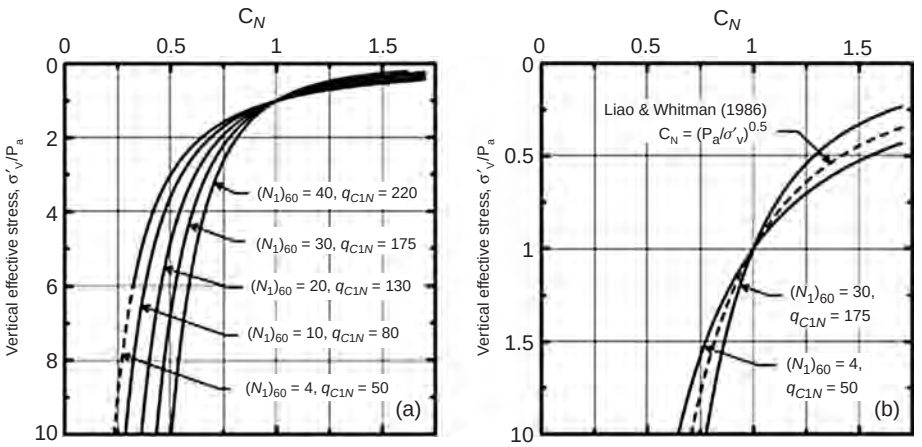


Figure 12.27 Overburden correction factor C_N for (a) vertical effective stresses up to 1000 kPa and (b) vertical effective stresses up to 200 kPa (Idriss and Boulanger (2008).

ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; C_S = correction for samplers with or without liners.

Boulanger and Idriss (2008) indicate that C_N should be calculated from:

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.784-0.0768\sqrt{(N_1)_{60}}} \leq 1.7 \tag{12.19}$$

with $(N_1)_{60}$ limited to values ≤ 46.

Where σ'_{vo} = effective vertical stress at the time of doing the SPT test (kPa); P_a = atmospheric pressure (100 kPa).

The equation for CPT is

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{1.338-0.249(q_{c1N})^{0.264}} \leq 1.7 \tag{12.20}$$

Figure 12.27 shows plots of the relationships.

The hammer energy ratio C_E varies between hammer types and for individual hammers and drill rig set ups. It varies between 0.5 and 1.0 for rope and pulley donut hammer, 0.7 and 1.2 for rope and pulley US Safety hammers and 0.8 and 1.3 for USA Safety hammers with automatic trip/free fall (Youd *et al.*, 2001). Tests on Australian free fall SPT hammers at Hume Dam surprisingly gave low C_E of about 0.6. It is clear that C_E should be measured for each project. If it is not, conservative assumptions will have to be made which may prove costly in overly conservative assessments of liquefaction potential.

Correction for borehole diameter (C_B) is seldom needed. $C_B = 1.0$ for holes 65–125 mm diameter. $C_B = 1.05$ for holes 150 mm diameter and $C_B = 1.15$ for holes of 200 mm diameter. Correction for rod length, C_R is necessary for rod length <3 m,

where $C_R = 0.75$. No other corrections are needed as they were not applied to develop the database.

SPT tests should be done with the split inner tubes in place, in which case $C_S = 1.0$. If the inner tube was left out, $C_S = 1.1$ to 1.3.

Idriss and Boulanger (2008) indicate that the correction for fines content, FC, (% passing 0.075 mm sieve) should be calculated from:

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \tag{12.21}$$

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right) \tag{12.22}$$

Boulanger and Idriss (2012) indicate that the $(N_1)_{60CS}$ values used in liquefaction assessments should be the mean values of the critical stratum, which is consistent with their treatment in analysing the case data.

Figure 12.28 shows the relationship with those used in Youd *et al.* (2001) and Cetin *et al.* (2004). It can be seen that the suggested correction is lower than used in Youd *et al.* (2001) but higher than Cetin *et al.* (2004). Idriss and Boulanger (2010) show that their suggested correction when applied to their database does not introduce any bias.

It is emphasised once again that users should not mix parts of one method with those from others because the analyses of the database of case histories has used the same inputs as recommended in the predictive methods.

12.5.3.6 *Evaluation of the Cyclic Resistance Ratio for M7.5 earthquake ($CRR_{7.5}$) from Cone Penetration Tests using the Idriss and Boulanger (2008) method*

The CRR from M7.5 earthquakes can be estimated from the curve in Figure 12.29. In this figure q_{c1N} is the normalized dimensionless cone penetration resistance calculated from:

$$q_{c1N} = C_Q(q_c/P_a) \tag{12.23a}$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \tag{12.23b}$$

where C_Q = normalizing factor for cone penetration resistance; σ'_{vo} = effective vertical stress at the time of doing the CPT test (kPa); P_a = atmospheric pressure (100 kPa); n = exponent that varies with soil type; and q_c = field cone penetration resistance measured at the tip in kPa.

The exponent n varies between 0.5 and 1.0 and can be estimated using the soil behaviour type index I_c as described in Section 12.5.2.1.

The correction to an equivalent clean sand value $(q_{c1N})_{cs}$ is done by

$$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N} \tag{12.24}$$

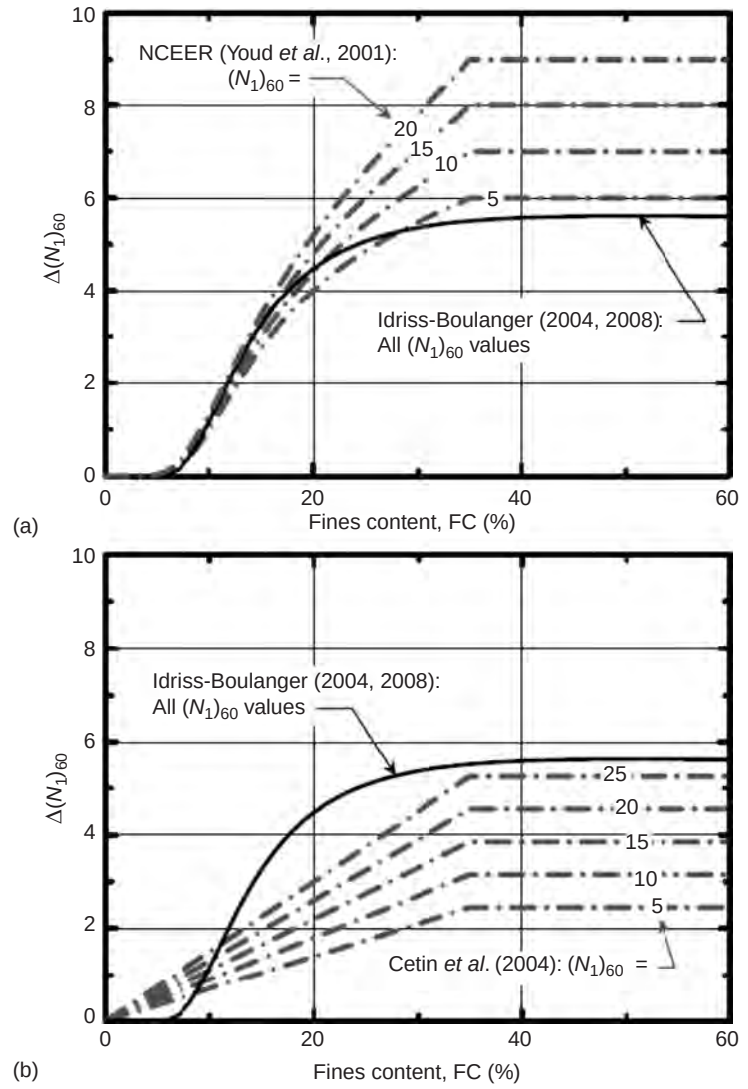


Figure 12.28 Variation of correction for fines content $\Delta(N_1)_{60}$ with fines content (% passing 0.075 mm sieve) (Idriss and Boulanger (2010).

where

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \cdot \exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right) \qquad (12.25)$$

Figure 12.30 shows this relationship for a number of q_{c1N}.

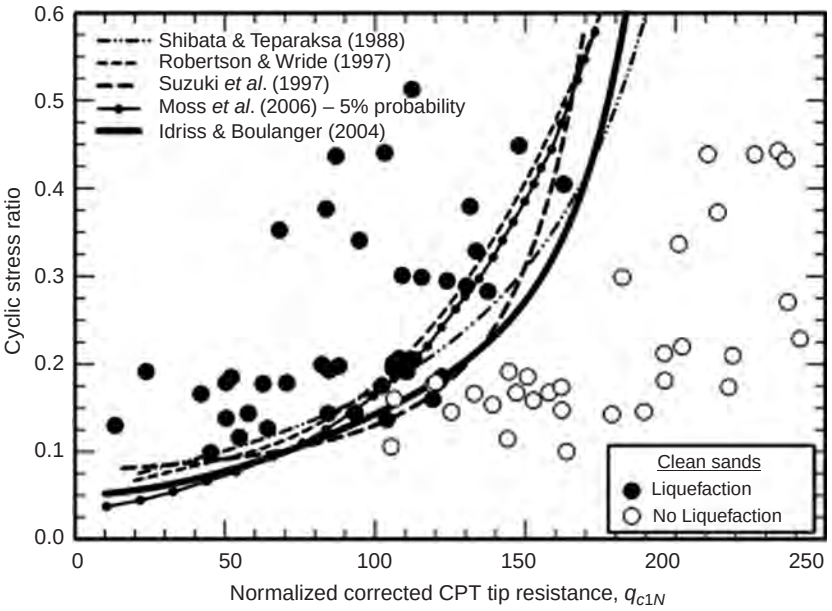


Figure 12.29 CPT clean-sand base Cyclic Resistance Ratio curve for magnitude 7.5 earthquakes and an effective overburden stress of 100 kPa with data from compiled case (Idriss and Boulanger, 2008).

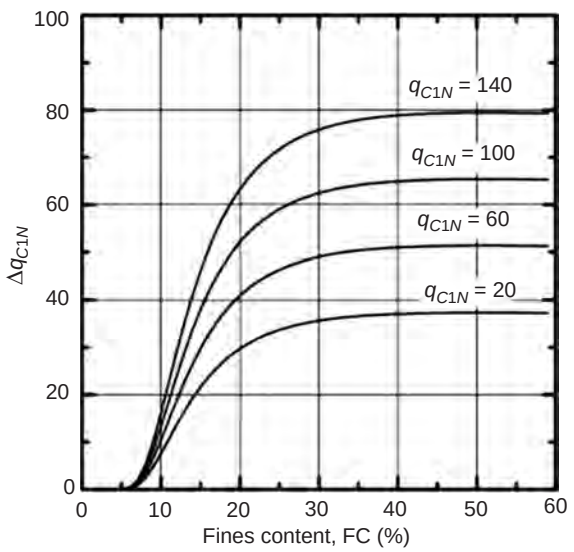


Figure 12.30 Variation of Δq_{c1N} with fines content (% passing 0.075 mm sieve) and q_{c1N}. (Idriss and Boulanger, 2008).

The equation for the Idriss and Boulanger curve is:

$$CRR_{7.5} = \exp\left(\frac{(q_{c1N})_{cs}}{540} + \left(\frac{(q_{c1N})_{cs}}{67}\right)^2 - \left(\frac{(q_{c1N})_{cs}}{80}\right)^3 + \left(\frac{(q_{c1N})_{cs}}{114}\right)^4 - 3\right) \quad (12.26)$$

Idriss and Boulanger (2008) do not state what probability the curve represents. They point out it is more conservative than the Robertson and Wride (1998) which was recommended in Youd *et al.* (2001). This reflects the larger database available to them.

Theoretical as well as laboratory studies indicate that CPT tip resistance is influenced by softer soil layers above or below the cone tip. As a result, measured CPT tip resistance is smaller in thin layers of granular soils sandwiched between softer layers than in thicker layers of the same granular soil. The amount of the reduction of penetration resistance in soft layers is a function of the thickness of the softer layer and the stiffness of the stiffer layers.

Vreugdenhil *et al.* (1994) first studied this, and Robertson and Fear (1995) and Youd *et al.* (2001) have modified that analysis to recommend the correction factor shown in Figure 12.31. Idriss and Boulanger (2008) have added the equation to model the Youd *et al.* (2001) correction factor. In this figure, H = thickness of the interbedded layer in mm, q_{cA} and q_{cB} = cone resistances of the stiff and soft layers, respectively and d_c = diameter of the cone in mm.

12.5.3.7 *Evaluation of Cyclic Resistance Ratio for M7.5 earthquake ($CRR_{7.5}$) from shear wave velocity using the Andrus and Stokoe (2000) method*

Shear wave velocity Vs, obtained from surface to down hole or by cross-hole testing, can be used to assess the CRR. This is particularly useful where the potentially liquefiable soils are gravelly and the gravel particles affect the SPT "N" value and prevent the penetration of a CPT. However the test may have difficulty locating thin, low velocity strata interbedded within the gravelly soils.

Idriss and Boulanger (2008) discuss the ability of shear wave methods to identify liquefiable soils and conclude that it is the least sensitive of the methods compared to SPT and CPT. They conclude that the method is best used to identify conditions in which the likelihood of liquefaction is high, low or uncertain.

The authors' experience is also that there is often not clear discrimination using this test and it is recommended that it be used only in conjunction with SPT and/or CPT data.

Nevertheless, the authors appreciate that there may be conditions such as sites with significant coarse gravels present that make the use of SPT or CPT or both impracticable and that shear wave velocity testing may be the only choice available.

The CRR for M7.5 earthquake can be estimated from the curve in Figure 12.32. In this figure Vs is corrected to an effective overburden stress of 100 kPa using:

$$V_{s1} = V_s\left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} \quad (12.27)$$

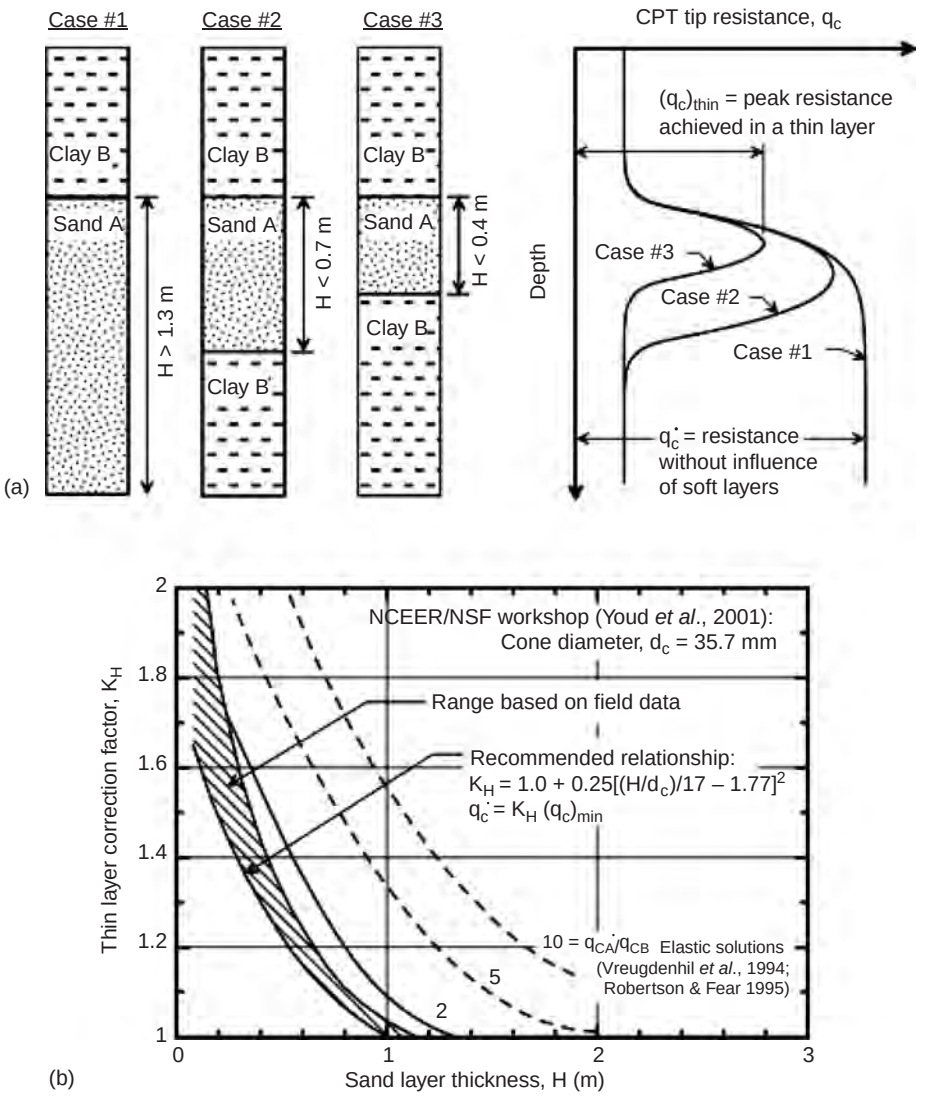


Figure 12.31 Thin-layer correction factor K_H for determination of equivalent thick-layer CPT resistance (a) a schematic of thin layer effects and (b) the relationship recommended by Youd *et al.* (2001) (Idriss and Boulanger, 2008).

where V_{s1} = overburden-stress corrected shear wave velocity, m/sec, P_a = atmospheric pressure, 100 kPa and σ'_{vo} = initial effective vertical stress in kPa.

The recommended lines in Figure 12.32 are dashed above $CRR_{7.5}$ of 0.35, to indicate field data is limited in this range, and do not extend below V_{s1} of 100 m/sec because there is a lack of data.

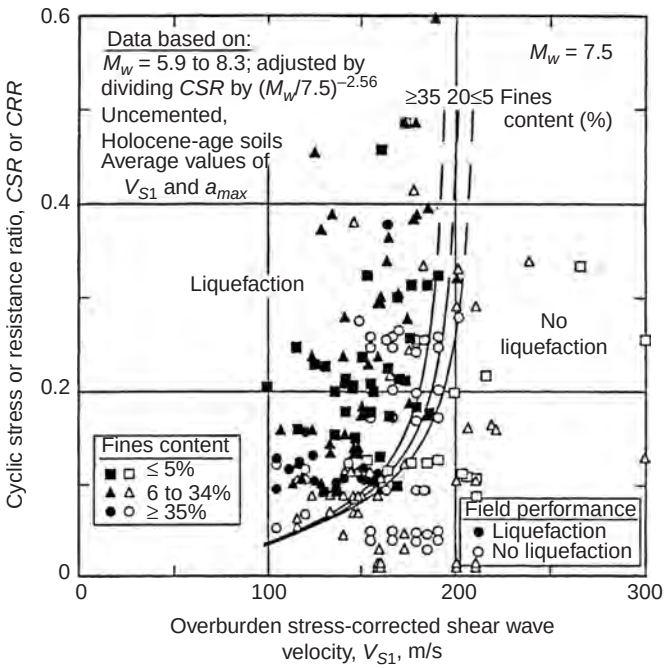


Figure 12.32 Shear wave clean sand base cyclic resistance ratio curve for magnitude 7.5 earthquake and an effective overburden stress of 100 kPa with data from compiled case histories (Youd *et al.*, 2001, from Andrus and Stokoe, 2000; reproduced with permission of ASCE).

12.5.3.8 *Earthquake magnitude scaling factors and factor of safety against liquefaction*

The procedures outlined in Sections 12.5.3.4 to 12.5.3.6 give $CRR_{7.5}$, Cyclic Resistance Ratios for M7.5 earthquakes.

Smaller magnitude earthquakes giving the same peak horizontal acceleration are less likely to initiate liquefaction, because the earthquake will have fewer cycles of motion. Larger magnitude earthquakes are more likely to initiate liquefaction. This is allowed for by using a magnitude scaling factor (MSF) in the equation for factor of safety (FS) against liquefaction:

$$FS = (CRR_{7.5}/CSR)MSF \tag{12.28}$$

where CSR = calculated cyclic stress ratio generated by the earthquake shaking and $CRR_{7.5}$ = Cyclic Resistance Ratio for magnitude 7.5 earthquakes.

Youd *et al.* (2001) present a discussion on MSF and conclude that the original Seed and Idriss (1982) factors are too conservative for $M < 7.5$.

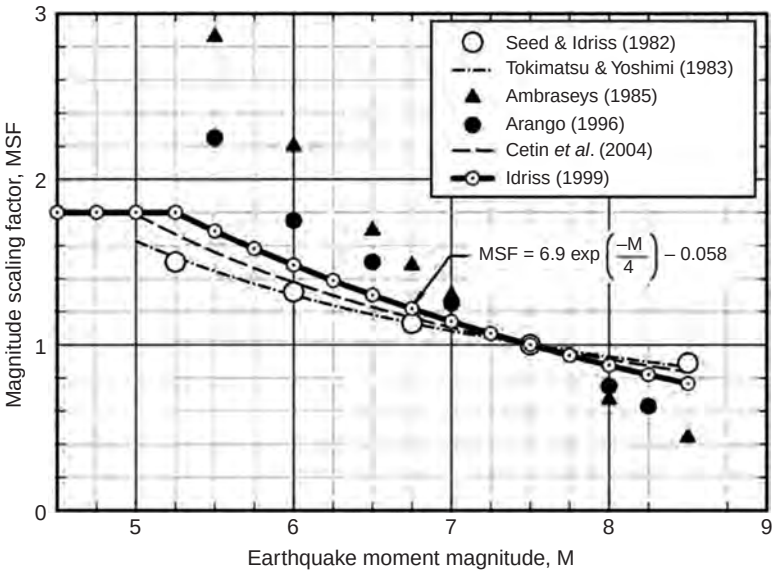


Figure 12.33 Magnitude scaling factors (MSF proposed by Idriss and Boulanger (2008) showing values proposed by other researchers.

Idriss and Boulanger (2008) discuss MSF and recommend use of the equations:

$$\mathrm{MSF} = 6.9 \exp\left(\frac{-\mathrm{M}}{4}\right) - 0.058 \tag{12.29}$$

$$\mathrm{MSF} \leq 1.8 \tag{12.30}$$

These are shown in Figure 12.33 along with scaling factors suggested by other researchers.

12.5.3.9 *Corrections for overburden stress and static shear stress*

The method of assessment of liquefaction potential described in Sections 12.5.3.2 to 12.5.3.7 is for horizontal or gently sloping ground and for depths less than about 15 m.

For assessments of liquefaction for embankment dams, the confining stresses may be higher than this and the dam will impose static shear stresses, which may alter the liquefaction potential.

Seed (1983) first proposed the factors K_σ and K_α to allow for the high overburden stresses and static shear stress respectively. They are applied by extending Equation 12.28 to:

$$\mathrm{FS} = (\mathrm{CRR}_{7.5}/\mathrm{CSR}) \cdot \mathrm{MSF} \cdot \mathrm{K}_\sigma \cdot \mathrm{K}_\alpha \tag{12.31}$$

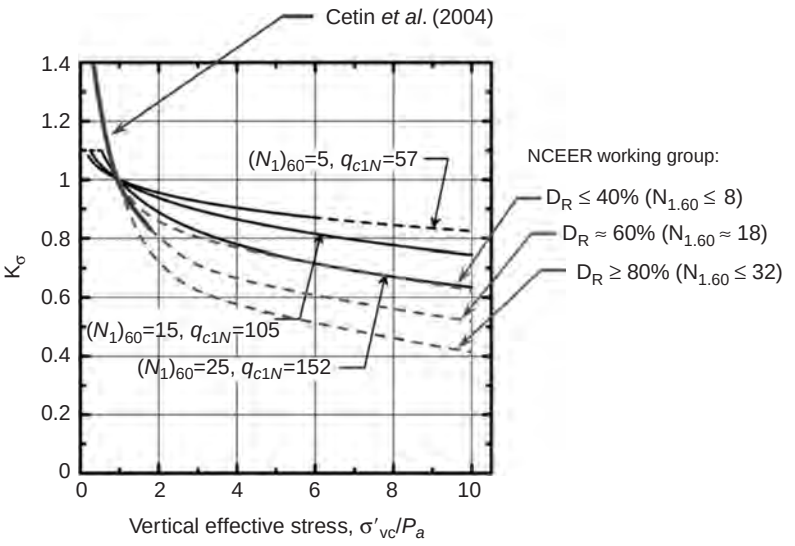


Figure 12.34 K_σ values for recommended by NCEER (Youd *et al.*, 2001); Idriss and Boulanger (2008), and Cetin *et al.* (2004). (Idriss and Boulanger (2010) (dashed lines are NCEER Working Group).

K_σ and K_α can be estimated as follows (Idriss and Boulanger, 2008):

Overburden stress factor K_σ

Youd *et al.* (2001), Seed *et al.* (2003) and Idriss and Boulanger (2008) use different methods to account for the overburden stress and to calculate K_σ.

(a) Youd *et al.* (2001) recommend the method of Hynes and Olsen (1999) which is based on laboratory data.
(b) Seed *et al.* (2003)/Cetin *et al.* (2004) used statistical regression analyses to develop criteria for effective stresses from 30 kPa to 175 kPa for SPT based assessments. For higher stresses they recommended the Hynes and Olsen (1999) method. These do not apply to CPT based assessments which are discussed in Moss *et al.* (2006) and have quite different K_σ values.
(c) Idriss and Boulanger (2008) recommended the method of Boulanger and Idriss (2004) which is based on laboratory tests data and theoretical considerations.

Figure 12.34 shows the three methods. It can be seen that the Idriss and Boulanger (2008, 2010) recommend somewhat larger values of K_σ than the other authors, particularly at higher stresses. It is one of the important distinctions between the methods.

The Idriss and Boulanger (2008) method is modelled by the following equations:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right) \leq 1.1 \tag{12.32}$$

Where the coefficient C_σ is given by;

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}} \leq 0.3 \tag{12.33a}$$

$$C_\sigma = \frac{1}{37.3 - 8.27(q_{c1N})^{0.264}} \leq 0.3 \tag{12.33b}$$

Idriss and Boulanger (2010) discuss the Cetin *et al.* (2004) and Moss *et al.* (2006) methods and argue they are inconsistent with each other and inconsistent with available experimental data.

It is suggested that the Idriss and Boulanger (2008) method be used but where K_σ values are critical to the outcome both the Idriss and Boulanger (2008) and Youd *et al.* (2001)/Hynes and Olsen (1999) values should be used, and where the Idriss and Boulanger (2008) values are controlling the outcome of the assessment of the likelihood of liquefaction, expert advice should be sought.

Sloping ground correction factor K_α

The CRR is affected by the presence of static shear stresses, such as exist within slopes or embankment dams. Idriss and Boulanger (2008) indicate that the case history data are insufficient to empirically determine this effect. They refer to the method of Harder and Boulanger (1997) which is based on laboratory tests and say the validity of these relationships for *in situ* sands is difficult to assess. Figure 12.35 presents that data.

Harder and Boulanger (1997) suggested the use of Figure 12.35 for soils with an effective confining stress less than 300 kPa. They point out that at significantly higher

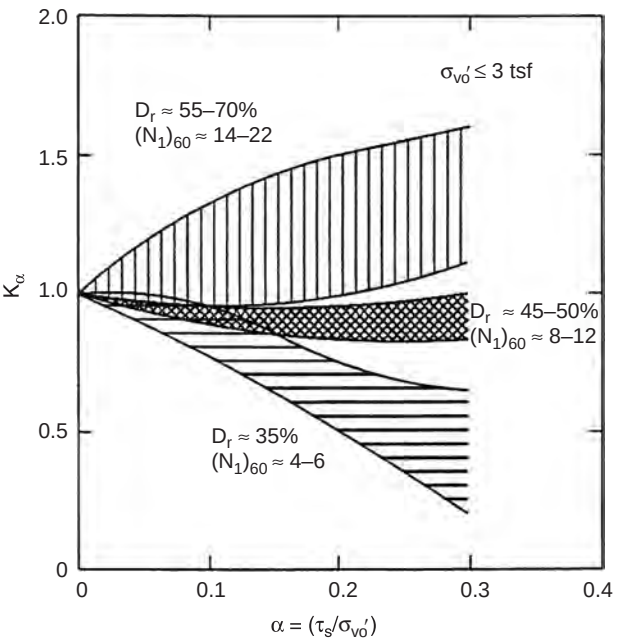


Figure 12.35 Sloping ground correction factor K_α suggested by Harder and Boulanger (1997).

confining stresses, sandy soils will be more contractive and low K_α may be appropriate. For high risk and critical projects on sloping ground or where there are high initial static shear stresses, they recommend site-specific laboratory testing using high quality samples.

In Figure 12.35:

$$\alpha = \tau_s / \sigma'_{vo} \tag{12.34}$$

where τ_s = static shear stress, σ'_{vo} = effective vertical stress.

The static shear stresses should be calculated from numerical analyses of the dam and its foundation.

Idriss and Boulanger (2008) present equations modelling the data in Figure 12.35 but the authors prefer readers to see the original plots so they gain an appreciation of the uncertainty in the data.

It is clear there is considerable uncertainty in the estimation of K_α. It seems that K_α will be less than 1.0 (meaning liquefaction is more likely) for relative densities lower than about 50%, so, marginal cases should be assumed liquefiable if there are existing shear stresses. If this becomes critical expert advice should be sought.

12.5.3.10 *Allowance for the age of the soil deposit*

There are a number of authors who have presented data to support the conclusion that the age of the soil deposit has an effect on the resistance of the soil to liquefaction. These include Youd and Perkins (1978), Seed (1979b), Arango *et al.* (2000), Leon *et al.* (2006), Andrus *et al.* (2009) and Hayati and Andrus (2009).

Hayati and Andrus (2009) analysed laboratory and field case histories and suggested the age of the deposit could be accounted for by using the equation:

$$CRR_K = CRR \times K_{DR} \tag{12.35}$$

where CRR_K = Cyclic Resistance Ratio corrected for age, CRR is the uncorrected Cyclic Resistance Ratio, and K_{DR} is the correction factor to capture the influence of age, cementation and stress history.

Figure 12.36 shows the plot of K_{DR} versus time since initial deposition or last critical disturbance including past liquefaction.

As pointed out by Hayati and Andrus (2009) an alternative interpretation of the data is for $K_{DR} = 1.0$ for Holocene (<10,000 years) and $K_{DR} = 1.8$ for Pleistocene (>10,000 years).

Andrus *et al.* (2009) present an alternative method based on measured to estimated shear velocity V_s. They include data which shows how the shear wave velocity V_s and CPT and SPT can give quite different assessments of liquefaction potential in Pleistocene and Tertiary sands.

It is suggested that these methods may be considered as additional information where SPT and CPT based methods are showing marginal potential for liquefaction in Pleistocene and Tertiary sands. However the methods are very approximate and not too much reliance should be made on them.

These methods should also be considered for application in assessing the liquefaction potential of recently deposited mine tailings and dredged fills, as there is evidence

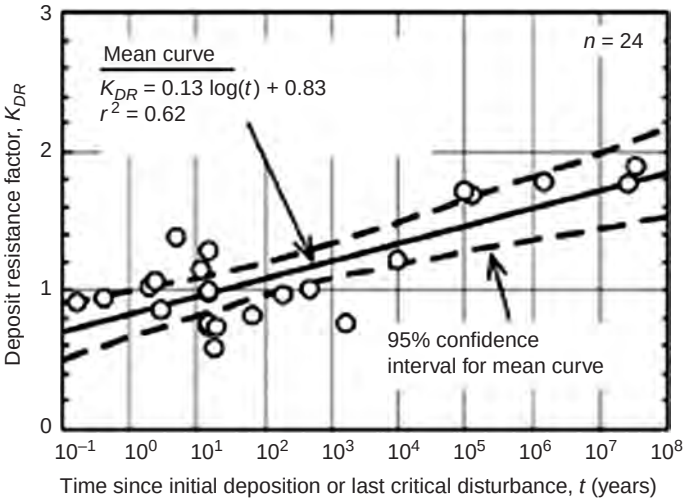


Figure 12.36 Plot of K_{DR} versus time since initial deposition or last critical disturbance including past liquefaction (Hayati and Andrus, 2009 with permission from ASCE).

the resistance to liquefaction may be lower than for the soils in the databases upon which the CRR are calculated.

12.6 LIQUEFIED UNDRAINED SHEAR STRENGTH AND POST EARTHQUAKE STABILITY ANALYSIS

12.6.1 Some general principles

As shown in Figure 12.9, undrained triaxial tests on saturated sands which display strain weakening behaviour, i.e. those susceptible to flow liquefaction, reach a steady state undrained strength, S_{us}.

In field situations, such soils may experience some pore pressure redistribution and the equivalent term is "liquefied shear strength $S_{u(LIQ)}$", (NSFW, 1998; Olsen and Stark, 2002). This is also referred to in the literature as the mobilized undrained critical strength (Seed and Harder, 1990; Seed, 2010) and undrained residual strength (Seed, 1987, Idriss and Boulanger, 2008).

To check the stability of a dam after the earthquake motion has ceased, a limit equilibrium stability analysis is carried out with zones which have been assessed as liquefied under the earthquake loading assigned the steady state undrained strength, S_{us}, or liquefied shear strength, $S_{u(LIQ)}$, depending on how the strength is assessed.

Saturated zones of the dam or foundation which are subject to limited strain softening (temporary liquefaction), or even strain hardening, will experience pore pressure build up in the earthquake which should be modelled in this "post earthquake stability analysis".

If the factor of safety is greater than 1.0 in these conditions, the slope will have experienced deformations during the earthquake, but no further deformation should

occur after the earthquake motion ceases apart from those resulting from dissipation of pore pressures. If however the factor of safety is less than 1.0, deformation will occur under gravity loading. In soils which are sufficiently loose and contractive, a flow slide may occur, with large, rapid deformations.

12.6.2 Background to the assessment of the liquefied shear strength $S_{u(LIQ)}$

The steady state undrained strength, S_{us}, of a soil is dependent on the void ratio, confining stresses, stress path of loading and soil fabric. The use of results from triaxial compression tests alone may be un-conservative, because S_{us} in triaxial extension and direct simple shear may be lower.

Castro (1975), Castro and Poulos (1977) and Poulos *et al.* (1985) developed a laboratory procedure using triaxial compression tests on reconstituted and high quality tube specimens. Corrections were made for volume changes that occur during sampling. However later studies, summarized in NSFW (1998) question the validity of this. NSFW (1998) recommend for high risk or high consequence of failure projects, laboratory testing using undisturbed samples obtained by ground freezing, but could not reach a consensus on test procedures.

As discussed in Idriss and Boulanger (2008), Seed *et al.* (2003) and Seed (2010) the shear strength that the liquefied soil mobilizes in the field is affected by void redistribution, particle intermixing and other field mechanisms which are not replicated in the laboratory.

As a result the only practical methods for estimating the liquefied shear strength $S_{u(LIQ)}$ is by back analysis of case studies and relating these to SPT and CPT data. However there are not a large number of cases available and often for these the SPT and CPT data is variable and sometimes of limited quality and/or the geometry of the embankment and the stratigraphy of the foundation is complicated. Olsen and Stark (2002) also describe some of these issues. As a result considerable judgement is exercised by authors in developing the methods, and the results are uncertain, with a wide range of strengths possible.

The issue is further complicated because there are two schools of thought:

- Those such as Seed *et al.* (2003), Seed (2010) who believe that the liquefied shear strength $S_{u(LIQ)}$ should be related to the SPT $(N_1)_{60CS}$ value directly with no allowance for the effective overburden stress. This is termed a "critical state" based method.
- Those such as Olsen and Stark (2002) who believe the residual strength ratio $S_{u(LIQ)}/\sigma'_{vc}$ should be used and related to the SPT $(N_1)_{60CS}$ or $(N_1)_{60}$ values. That is the residual strength is controlled by the effective overburden stress. Idriss and Boulanger (2008) and Robertson (2010) have also developed such methods.

There is no consensus on which of the two approaches is best. Idriss and Boulanger (2008) provide both methods without guidance on which to adopt but say they believe the residual strength ratio method provides a better representation of the potential effects of void redistribution than the liquefied residual strength, while it is recognized

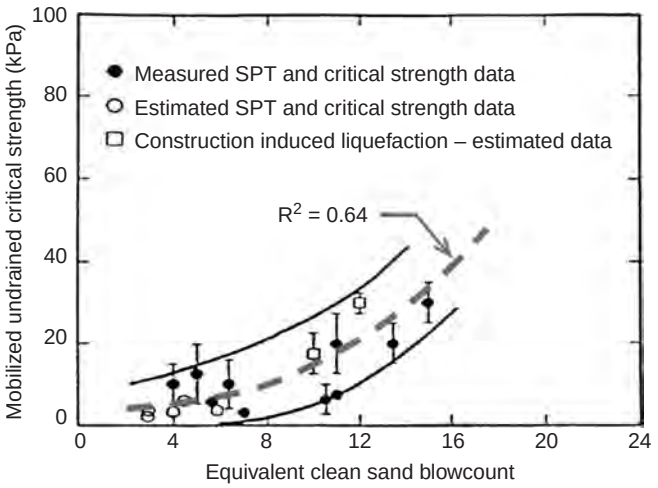


Figure 12.37 Liquefied shear strength $S_{u(LIQ)}$ versus SPT $(N_1)_{60CS}$ value according to Seed (2010; Seed and Harder, 1990).

that neither correlation fully accounts for the numerous factors that influence void redistribution processes.

12.6.3 Some methods for assessing the strength of liquefied soils in the embankment and foundation

12.6.3.1 *"Critical State" based methods*

Figure 12.37 shows the Seed and Harder (1990) method upon which Seed (2010) has added a line of best fit which he suggests is used.

Figure 12.38 shows the Idriss and Boulanger (2008) method which is based on case studies from Seed (1987), Seed and Harder (1990), and Olsen and Stark (2002). They provide a second plot for CPT data. Note that this requires the median $(N_1)_{60cs\text{-}Sr}$ value for the liquefied strata. The corrections for fines content for calculating the $(N_1)_{60cs\text{-}Sr}$ value is based on Seed (1987) and this is different to the fines corrections used for the liquefaction triggering correlations discussed in Section 12.5.3.5 (Equation 12.21). The correction for fines content for estimating $S_{u(LIQ)}$ is based on:

$$(N_1)_{60CS\text{-}Sr} = (N_1)_{60} + \Delta(N_1)_{60\text{-}Sr} \tag{12.36}$$

where $\Delta(N_1)_{60\text{-}Sr}$ are based on the values given in Table 12.6 from Seed (1987).

The recommended curves for the two methods are not greatly different with Idriss and Boulanger (2008) being a little more conservative.

12.6.3.2 *Normalized strength ratio methods*

Figures 12.39 and 12.40 show the Olsen and Stark (2002) and Idriss and Boulanger (2008) methods for SPT and for CPT data. σ'_{vo} is the pre-failure vertical effective stress for CPT and SPT.

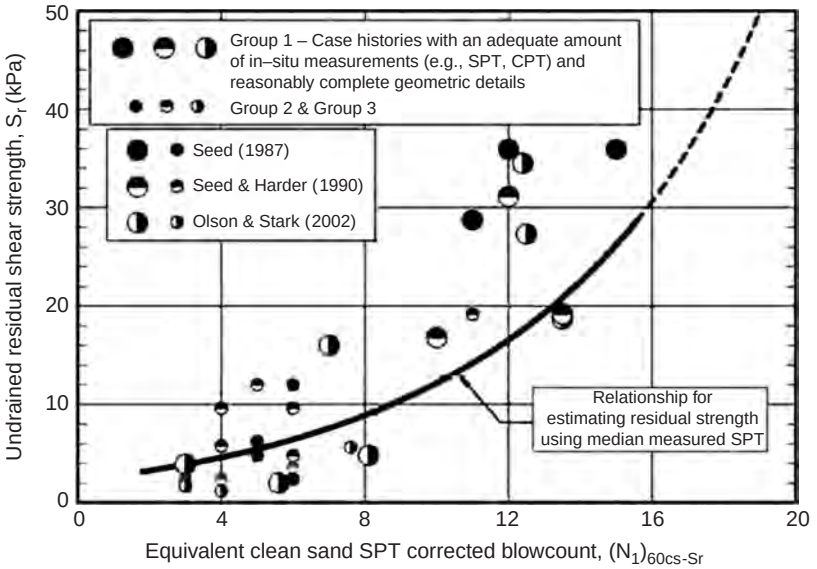


Figure 12.38 Liquefied shear strength $S_{u(LIQ)}$ versus SPT $(N_1)_{60CS}$ value according to Idriss and Boulanger (2008).

Table 12.6 Values of $\Delta(N_1)_{60\text{-}Sr}$ for fines correction for estimating $S_{u(LIQ)}$ from Seed (1987).

Fines content (% passing 0.075 mm)	$\Delta(N_1)_{60\text{-}Sr}$
10	1
25	2
50	4
75	5

Olsen and Stark (2002) suggest the central dashed lines may be used for design. These are described by:

$$S_{u(LIQ)}/\sigma'_{vo} = 0.03 + 0.0143\,(q_{ci}) \pm 0.03 \quad \text{for } q_{c1} < 6.5\,\text{mPa} \tag{12.37}$$

and

$$S_{u(LIQ)}/\sigma'_{vo} = 0.03 + 0.0075((N_1)_{60}) \pm 0.03 \quad \text{for } (N_1)_{60} < 12 \tag{12.38}$$

Note the Olsen and Stark (2002) correlations for SPT and CPT uses the $(N_1)_{60}$ and (q_{c1}) values which are not corrected for the fines content of the soil whereas the Idriss and Boulanger (2008) correlations use the equivalent clean sand SPT and CPT values $(N_1)_{60\text{-}Sr}$ and $(q_{c1N})_{cs\text{-}Sr}$.

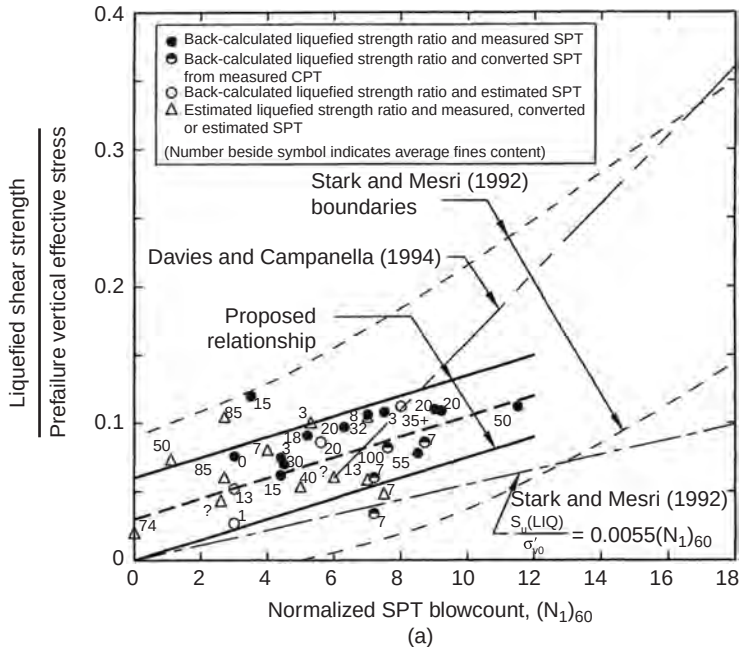

Back-calculated liquefied strength ratio and measured SPT
Back-calculated liquefied strength ratio and converted SPT from measured CPT
Back-calculated liquefied strength ratio and estimated SPT
Estimated liquefied strength ratio and measured, converted or estimated SPT
(Number beside symbol indicates average fines content)
Stark and Mesri (1992) boundaries
Davies and Campanella (1994)
Proposed relationship
Stark and Mesri (1992)
$\frac{S_{u(LIQ)}}{\sigma'_{vo}} = 0.0055(N_1)_{60}$
Liquefied shear strength / Prefailure vertical effective stress
Normalized SPT blowcount, $(N_1)_{60}$
(a)

Figure 12.39(a) Liquefied undrained strength ratio $S_{u(LIQ)}/\sigma'_{vo}$ versus SPT $(N_1)_{60}$ value (Olsen and Stark, 2002).

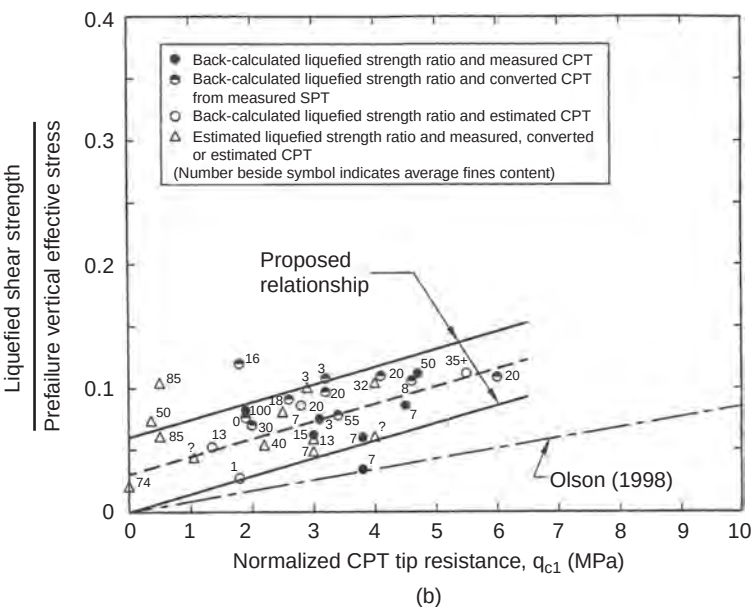

Back-calculated liquefied strength ratio and measured CPT
Back-calculated liquefied strength ratio and converted CPT from measured SPT
Back-calculated liquefied strength ratio and estimated CPT
Estimated liquefied strength ratio and measured, converted or estimated CPT
(Number beside symbol indicates average fines content)
Proposed relationship
Olson (1998)
Liquefied shear strength / Prefailure vertical effective stress
Normalized CPT tip resistance, q_{c1} (MPa)
(b)

Figure 12.39(b) Liquefied undrained strength ratio $S_{u(LIQ)}/\sigma'_{vo}$ versus CPT q_{c1} value (Olsen and Stark, 2002).

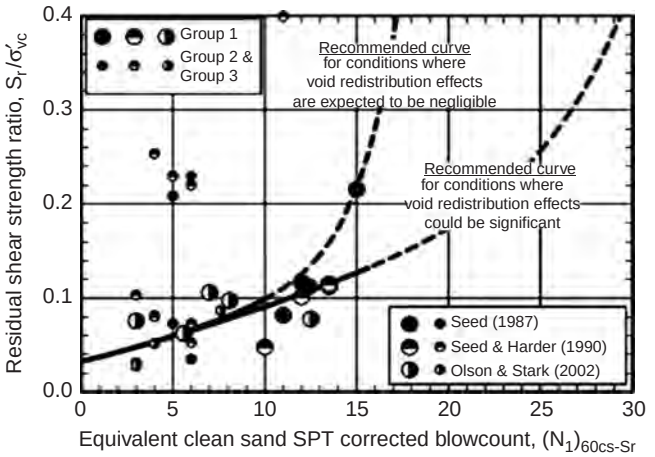

Residual shear strength ratio, S_r/σ'_{vc}
Group 1
Group 2 & Group 3
Recommended curve for conditions where void redistribution effects are expected to be negligible
Recommended curve for conditions where void redistribution effects could be significant
Seed (1987)
Seed & Harder (1990)
Olson & Stark (2002)
Equivalent clean sand SPT corrected blowcount, $(N_1)_{60cs\text{-}Sr}$

Figure 12.40(a) Liquefied undrained strength ratio $S_{u(LIQ)}/\sigma'_{vc}$ versus SPT $(N_1)_{60\text{-}Sr}$ value (Idriss and Boulanger, 2008).

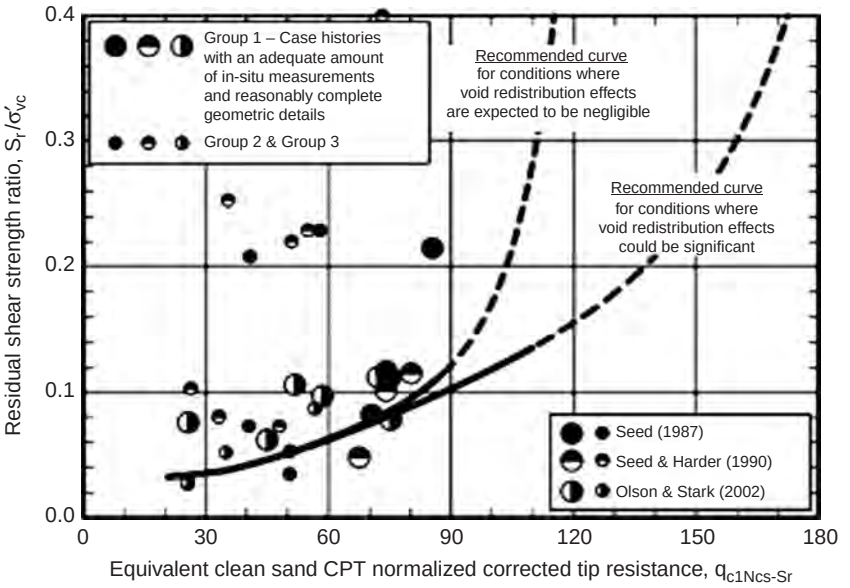

Residual shear strength ratio, S_r/σ'_{vc}
Group 1 – Case histories with an adequate amount of in-situ measurements and reasonably complete geometric details
Group 2 & Group 3
Recommended curve for conditions where void redistribution effects are expected to be negligible
Recommended curve for conditions where void redistribution effects could be significant
Seed (1987)
Seed & Harder (1990)
Olson & Stark (2002)
Equivalent clean sand CPT normalized corrected tip resistance, $q_{c1Ncs\text{-}Sr}$

Figure 12.40(b) Liquefied undrained strength ratio $S_{u(LIQ)}/\sigma'_{vc}$ versus CPT $(q_{c1N})_{cs\text{-}Sr}$ value (Idriss and Boulanger, 2008).

The Idriss and Boulanger (2008) line for $< (N_1)_{60\text{-}Sr} < 15$ plots a little lower although the plots are not directly comparable because Olsen and Stark (2002) use $(N_1)_{60}$ and Idriss and Boulanger (2008), $(N_1)_{60\text{-}Sr}$.

Seed (2010) strongly criticizes the Idriss and Boulanger (2008) method, in particular the "recommended curve for conditions where void ratio redistribution effects are expected to be negligible". He argues that three of the data points use incorrect and

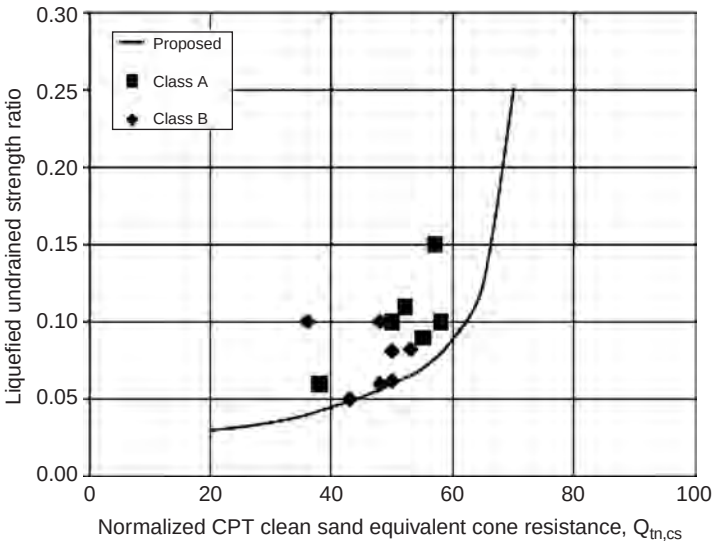


Figure 12.41 Liquefied shear strength ratio and normalized CPT clean sand cone resistance $Q_{1n,cs}$. Class A and B refer to the quality of data points. (Robertson, 2010 with permission from ASCE).

over stated $S_{u(LIQ)}/\sigma'_{vc}$ ratios. He argues that the case data does not support this curve. Youd (2010) discusses this and says that Idriss and Boulanger unequivocally state that the upper curve is based on laboratory data and was not guided by the points with incorrect strength in any way. He also says that a transformation from contractive to dilative behaviour occurs at about a corrected SPT blow count of 15, and that should be accompanied by a major increase in shear strength.

12.6.3.3 *Other methods*

Robertson (2010) presents Figure 12.41 which is based on case histories. The limiting value of liquefied undrained strength ratio is assumed to be 0.4 at $Q_{1n,cs} = 70$. This is based on the soils being dilative at $Q_{1n,cs} > 70$ so not being subject to flow liquefaction.

Baziar and Dobry (1995), Ishihara (1993) and Cubrinovski and Ishihara (2000a, b) have investigated the boundary of flow liquefaction conditions. Figure 12.42 summarizes the outcomes. These support the use of larger liquefied strengths for soils with high $(N_1)_{60}$ values at least above $(N_1)_{60} = 15$.

12.6.4 Some other factors to consider

NSFW (1998) caution against the use of $(S_{u(LIQ)}/\sigma'_{vo})$ ratios, particularly for clean sands but indicated most at the workshop accepted the use for silty sand soils. Olsen and Stark (2003) indicate that, in their view, at least for silty sands with >12% fines, it is reasonable to allow for the increase in $S_{u(LIQ)}$ which would be indicated by the increase in σ'_{vo} from the berm. For high risk projects they suggest laboratory consolidation tests

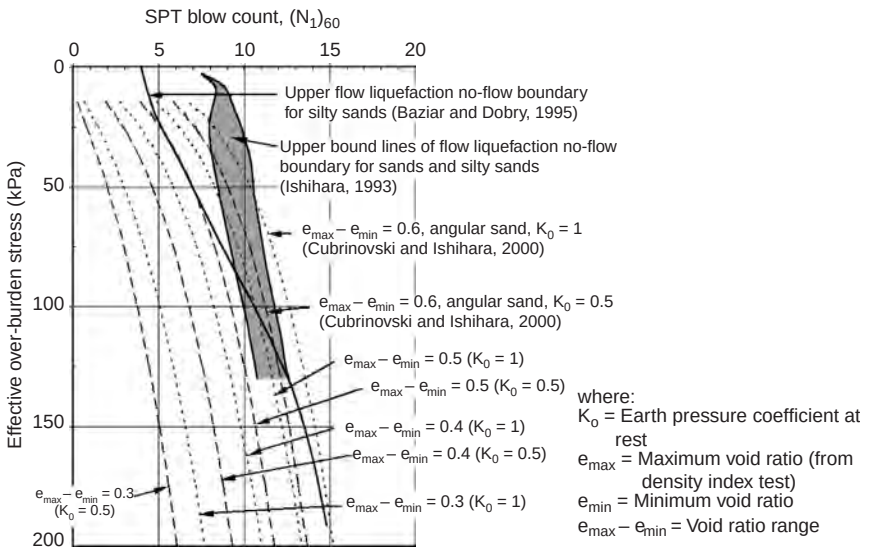

SPT blow count, $(N_1)_{60}$
0
5
10
15
20
0
50
100
150
200
Effective over-burden stress (kPa)
Upper flow liquefaction no-flow boundary for silty sands (Baziar and Dobry, 1995)
Upper bound lines of flow liquefaction no-flow boundary for sands and silty sands (Ishihara, 1993)
$e_{max} - e_{min}$ = 0.6, angular sand, K_0 = 1 (Cubrinovski and Ishihara, 2000)
$e_{max} - e_{min}$ = 0.6, angular sand, K_0 = 0.5 (Cubrinovski and Ishihara, 2000)
$e_{max} - e_{min}$ = 0.5 (K_0 = 1)
$e_{max} - e_{min}$ = 0.5 (K_0 = 0.5)
$e_{max} - e_{min}$ = 0.4 (K_0 = 1)
$e_{max} - e_{min}$ = 0.4 (K_0 = 0.5)
$e_{max} - e_{min}$ = 0.3 (K_0 = 0.5)
$e_{max} - e_{min}$ = 0.3 (K_0 = 1)
where:
K_o = Earth pressure coefficient at rest
e_{max} = Maximum void ratio (from density index test)
e_{min} = Minimum void ratio
$e_{max} - e_{min}$ = Void ratio range

Figure 12.42 Comparison of flow liquefaction boundaries in terms of SPT $(N_1)_{60}$ for sands and silty sands from monotonic laboratory undrained tests and earthquake triggered field case (Hunter and Fell, 2003a, b).

be carried out to confirm that it is parallel to the steady state line which is implicit in the assumption that $S_{u(LIQ)}/\sigma'_{vo}$ is constant.

Idriss and Boulanger (2008) favour the stress ratio approach but do not give any clear direction on what to do in regards designing berms for remedial works. Seed *et al*. (2003) favour the critical state based methods which unless consolidation occurs under the loads of a berm would result in no strength increase.

This is particularly important if it is planned to use a berm to improve the post earthquake stability of a dam. Based on the points above it would appear unwise to allow for the strength increase from the increased effective stress of a berm to be constructed for remedial works for clean sands, and the $S_{u(LIQ)}$ obtained without the berm should be assumed to apply after the berm is built, unless sampling and laboratory testing indicates otherwise.

If strength increase was to be allowed for e.g. in silty sands it would also be wise to confirm the strength increase by carrying out SPT or CPT through the berm after it is constructed.

12.6.5 Recommended approach to assessing the liquefied undrained strength soils of in the embankment and foundation

Based on the above the following is recommended:

(a) Use both critical state and normalised strength ratio methods and apply equal weighting to each method.

(b) Either the Idriss or Boulanger (2008), or Seed and Harder (1990)/Seed (2010) or Olsen and Stark (2002) methods may be used. These may be compared to the Robertson (2010) method.
(c) Use SPT and CPT tests wherever practicable. Give equal weighting to each or weight towards the test giving the lower strength.
(d) In the authors' experience the majority of foundation liquefaction cases will satisfy the requirements for Idriss and Boulanger (2008) "recommended curve for where void redistribution effects could be significant" because the liquefiable layer is overlain by a lower permeability strata. In view of this and the reservations of Seed (2010) it is suggested that only this curve (not the upper one) be used unless expert advice is sought.
(e) Do not allow for any increase in the liquefied undrained strength beneath a berm which is to be added to improve post earthquake stability unless the requirements of Section 12.6.4 are met. If a berm has already been constructed carry out SPT and CPT in the liquefiable soils below the berm and use these data to assess the liquefied undrained strength.
(f) Take account of the fact that there are significant uncertainties in these methods and use strengths between the best estimate and lower bound strengths.

If the liquefied undrained strength of such soils is controlling the assessment of the safety of the dam, and large costs are potentially involved in remedial works, it may be warranted to seek advice from one of the experts in the field.

12.6.6 Methods for assessing the post earthquake strength of non-liquefied soils in the embankment and foundation

12.6.6.1 *Saturated potentially liquefiable soils*

Saturated soils which are potentially liquefiable, but which have not been subject to sufficient cyclic shear stress to reach a liquefaction condition, will develop positive pore pressures during the cyclic loading of the earthquake. The amount of positive pore pressure will depend on many factors relating to the soil and the cyclic loading. These pore pressures should be allowed for in post earthquake stability analyses.

Idriss and Boulanger (2008) refer to Marcuson *et al.* (1990) for level ground conditions but warn that pore pressures will be higher for situations where static shear stresses are present. The Marcuson *et al.* (1990) data is shown in Figure 12.43 and relates the residual excess pore pressure ratio R_u, at the end of the earthquake loading to the factor of safety against liquefaction, FS calculated from Equation 12.28.

If these pore pressures become critical, and the project is sufficiently important, seek expert advice. For more critical projects, carefully planned and executed cyclic laboratory tests should be used.

12.6.6.2 *Cyclic softening in clays and plastic silts*

Boulanger and Idriss (2006) and Idriss and Boulanger (2008) describe how to estimate the effects of cyclic softening of clays and plastic silts. The discussion relates

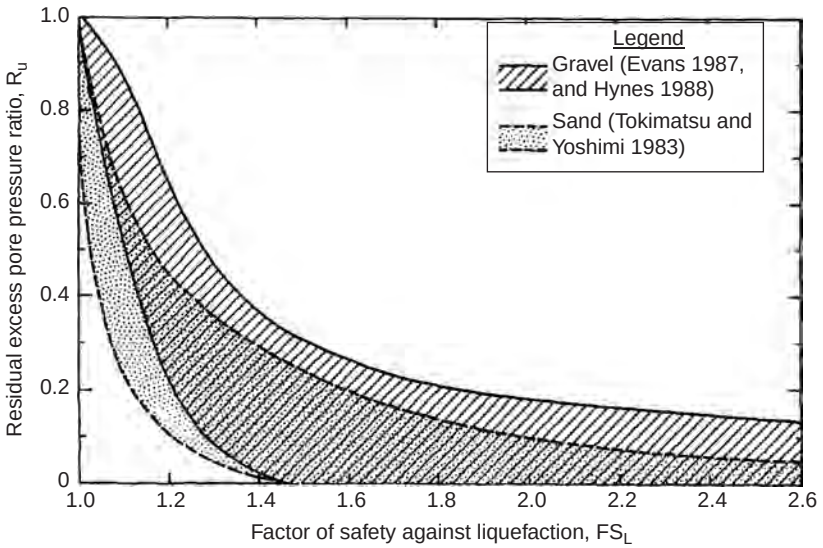


Figure 12.43 Method for approximate estimation of the residual excess pore pressure R_u in non liquefied zones (Marcuson *et al.*, 1990).

to saturated soils with over-consolidation ratios of 1 to 4. Such soils do occur in association with liquefiable soils, and may also occur as mine tailings.

The boundary between which this behaviour and liquefaction occurs is discussed in Section 12.5.2.1 (iv).

12.6.6.3 *Compacted plastic and non-plastic soils*

It is suggested that:

(a) For clay soil zones in the embankment and foundation assign strength and pore pressures consistent with the soil's behaviour in static loading after being cracked and disturbed by the earthquake. If the clay is contractive in nature, use undrained strengths. If it is dilative on shearing, use effective stress strengths c', ϕ'. Usually there will be some cracking and loosening; if so fully softened strengths, would apply (e.g. $c' = 0$, $\phi' = \phi'_{peak}$). Some apply an arbitrary 10% or 15% loss of strength.

(b) For well compacted free draining rockfill filters and dense sands and gravels adopt the effective stress strengths c', ϕ', with no change in the pore pressures. If large deformations are expected in the earthquake the dense granular materials may have loosened and will have a strength approaching the critical state strength, rather than the peak strength. In practice this can be accommodated by a small reduction from the expected peak strengths.

12.6.7 Liquefaction potential and limit equilibrium stability analysis

The analysis of liquefaction potential is best carried out as follows:

- Assemble the SPT and CPT /CPTU data for potentially liquefiable strata into spreadsheets. In so doing assess the data for quality and inconsistencies and remove or "tag" as unreliable poor quality or inconsistent data.
- For SPT include all "N" in the spreadsheet. For CPT/CPTU enter data at 50 mm intervals as recorded.
- Analyse the data to determine if the soils are potentially liquefiable using the criteria described above and summarized in Section 12.5.2.1. Exclude from further considerations soils which are not liquefiable, or which are above the water table and will not be saturated under any condition.
- For the potentially liquefiable strata, assess the factor of safety against liquefaction for a number of peak ground accelerations within the range to be assessed for the site. Allow for amplification effects of the ground motions when applying the simplified method, or estimate the cyclic stresses at the potentially liquefiable layer using dynamic analyses of the embankment.
- From these data, plot plans and sections showing contours of the peak ground accelerations and associated Annual Exceedance Probability (AEP) to trigger liquefaction of the liquefiable zones.

The strength of liquefied strata and other strata should be estimated as follows:

- For the strata predicted to liquefy (i.e. FS for liquefaction < 1.0), estimate the liquefied undrained strength. There are often several different liquefiable strata with different properties.
- For saturated non-liquefied non-plastic strata, estimate the residual pore pressure built up during the earthquake. For saturated plastic strata which are subject to cyclic softening, estimate the effects of cyclic softening. For compacted fills allow for the effects of cracking and deformation. These will be earthquake loading dependent (refer to Section 12.7).
- If there is strain weakening over-consolidated clay in the foundation, estimate the strain weakened strength allowing for the earthquake load being considered and potential deformations in the strain weakening strata.
- The post earthquake stability analysis is done with conventional limit equilibrium analysis methods. No loading from the earthquake is applied, since this is a post earthquake analysis. The sensitivity of the factors of safety to the extent of liquefied zones, and the uncertainty in the strength inputs should be assessed.

12.6.8 Site investigations requirements and development of geotechnical model of the foundation

The following are some recommendations regarding site investigations of foundations of dams for assessing the likelihood and consequences of liquefaction.

(a) *Review of available data.*

Gather together all available data on the foundations, including for existing dams, site investigations and laboratory testing carried out prior to construction, geological mapping, reports, and photographs taken during construction, monitoring data and site investigations and laboratory testing carried out since the dam was constructed.

(b) *Preliminary geotechnical model and planning site investigations.*

Draw plans and sections of the available data, taking account of the depositional environment of the foundation soils.

From this develop an interpretive geotechnical model of the strata in the foundation, and summarize the properties in a form useful for liquefaction assessment.

Assess the adequacy of the available data and plan site investigations to supplement this data.

(c) *Site investigation methods.*

Foundations should be investigated by both boreholes in which SPT tests are carried out on potentially liquefiable soils, and undisturbed thin wall tube samples taken of other strata; and by Cone Penetration tests (CPT, CPTU). This is necessary because CPT are best able to detect layering in the strata, but SPT and laboratory tests are required to provide data for some parameters and to make available two methods for assessing the liquefaction potential and post earthquake liquefied strengths.

Some of these (SPT and CPT) should be done adjacent each other for correlation purposes.

Boreholes should where practicable be drilled using wash boring methods with casing and drilling mud support. If hollow flight augers are used (as may be necessary for holes drilled through an existing dam) take measures to prevent "blowing" of the strata at the base of the augers with subsequent reduction in SPT blow counts.

Caution should be used in interpretation of SPT data where sonic drilling methods are used as the sonic vibrations could lead to densification of loose sand materials below the drill bit.

The authors prefer to confirm SPT data in sonic drill holes with SPT in adjacent boreholes drilled using conventional methods and drilling mud, subject to it being acceptable to drill using wash boring without risks of hydraulic fracture of the core of the embankment.

Take samples from all SPT tests in potentially liquefiable strata and test for moisture content and fines content (% passing 0.075 mm sieve). For representative samples test for Atterberg limits.

In stratified strata, ensure different soil types that are collected in the SPT samples are carefully separated and tested. For critical projects, the energy efficiency of the SPT hammer being used in the investigations should be measured. If it is not measured assume 60% efficiency even for free fall drop hammers.

Record the level of the water table during drilling and preferably install piezometers to monitor groundwater pressures at the time of the investigations and as they fluctuate with river flows.

For SPT tests in gravelly soils take blow counts for each 75 mm as an aid to detecting interference by gravel particles and allow adjustment to allow for this. For gravelly soils consider the use of shear wave velocity in addition to boreholes with SPT.

For CPT and CPTU tests in layered soils adjust the data for the effects of soft and stiff layers as detailed in Figure 7 of Youd *et al.* (2001). CPTU is preferred over CPT as the measured dynamic pore pressures provide an indication of the contractive vs dilative behaviour of the soil under dynamic loading conditions.

(d) *Geotechnical model*

Draw plans and sections of the available data including the data from the new investigations, taking account of the depositional environment of the foundation soils.

From this develop an updated interpretive geotechnical model of the strata in the foundation, and summarize the properties in a form useful for liquefaction assessment. Take particular attention to the continuity and aerial extent of potentially liquefiable strata.

12.7 SEISMIC DEFORMATION ANALYSIS OF EMBANKMENT DAMS

12.7.1 Preamble

In Fell *et al.* (2005) this section was titled "Seismic stability analysis of embankments". That was in keeping with most references at the time. As described above most embankments will, under large seismic loads, yield during part of the loading cycle, resulting in permanent deformations. However that does not mean the dam has "failed" provided the deformations are tolerable. Hence the emphasis in this Section is on deformations, not stability.

It is only dams which experience liquefaction, or are founded on strain weakening soils resulting in post earthquake factors of safety below 1.0 that have a stability issue.

12.7.2 Performance of embankment dams during earthquakes

Major reviews of past performance have been conducted by Sherard (1967), Sherard *et al.* (1974), Seed (1979b, 1983) and Seed *et al.* (1978, 1985b). These studies show that based on the data available at that time:

- Even at short distances from the epicentres, there have been no complete failures of embankments built of clay soils, but several dams have come close to failure.
- Well constructed dams of clay soils on clay or rock foundation, not susceptible to strain weakening, can withstand extremely strong shaking resulting from earthquakes of up to Magnitude 8.25 with peak ground accelerations ranging from 0.35 g to 0.8 g (They suffer cracking but few, if any, have failed catastrophically).
- Dams which have suffered complete failure as a result of earthquake shaking have been constructed primarily with saturated sandy materials or on saturated

sand foundations. Liquefaction is a major contributory factor in these failures. The dams most susceptible to failure under earthquake loading are hydraulic fill dams and tailings dams constructed using upstream methods, because they are susceptible to liquefaction if the fill or tailings are granular and saturated.

– In dams constructed of saturated cohesionless soils the primary cause of damage or failure is the build up of pore water pressure (liquefaction) under the earthquake loading.
– There are very few cases of dam failures during the earthquake shaking. Most of the failures occur from a few minutes up to twenty-four hours after the earthquake. However, cracking and displacements do occur during the earthquake.

The following suggested approach is based on the authors experience and relies on risk-based logic. This is considered applicable because the earthquake loadings are often expressed in probabilistic terms and, even if they are expressed in deterministic terms, there is a significant uncertainty to those loads. As well as this approach the methods for estimating deformations are often approximate and even the most sophisticated methods have a range of possible outcomes.

12.7.3 The methods available and when to use them

There are a number of methods available for estimating the deformations which may occur in embankments and their foundations during and post seismic loading. These can be summarized as:

- *Screening and empirical database methods*. These are applicable to embankments and their foundations which do not liquefy, or experience significant loss of strength due either to build up of pore pressure or strain weakening. These should only be applied if the criteria listed in Section 12.7.4 are satisfied, and should only be relied upon if the estimated deformations are much less than tolerable, e.g. crest settlements are much less than the available freeboard.
- *Simplified methods for estimating deformations during earthquakes*. These methods are also only applicable to embankments and their foundations which do not liquefy. They assume the post-earthquake deformations are negligible and the deformations during the earthquake are due to the action of the horizontal inertia forces induced by the earthquake. They are commonly based on the Newmark (1965) principles.
- *Post earthquake deformations for liquefied conditions*. These methods assume that the deformations are primarily caused by gravitational forces acting on an embankment following the earthquake and allow for the reduction in strength caused by liquefaction or strain weakening of other soils. They may be carried out using limit equilibrium and or static numerical methods.
- *Advanced numerical methods for estimating deformations during and post earthquake for non-liquefied and liquefied conditions*. These cover a wide range of sophistication and complexity and include dynamic analyses using total and effective stress methods, linear and non-linear models and varying degrees of refinement of how pore pressures are developed and coupled to deformations.

Perlea and Beaty (2010) give an overview of the methods and their application.

These methods vary considerably in their degree of sophistication and time to do the analyses. In view of this often a staged approach to estimating deformations is recommended.

It should not be assumed that the screening, database and simplified methods are conservative and where estimated deformations are near to the tolerable deformations more advanced methods should be used or measures taken to reduce deformations. That is the principle behind the suggested approach described in Section 12.7.4.

Where significant strain weakening of over-consolidated high clay content soils within the dam foundation or in the embankment may occur due to displacements induced by the earthquake, post earthquake stability should be analysed taking account of the strain weakening.

The screening, database and simplified methods are not applicable to tailings dams constructed from hydraulically placed tailings because the tailings are not compacted to the degree required for conventional earthfill and rockfill, and will as a result be more subject to densification during seismic loading, and may be subject to pore pressure generation and liquefaction.

The authors also emphasise that the post earthquake internal erosion and piping risk must not be ignored as it may be a critical condition. This is discussed in Section 8.3.2.11.

12.7.4 Suggested approach to estimation of deformations

The estimation of the deformations which may occur in an embankment and its foundations due to seismic loading should be assessed as follows:

(1) Assess whether the embankment or its foundation are susceptible to liquefaction using the methods described in Section 12.5.

(2) For embankments and foundations which are not susceptible to liquefaction or experiencing significant pore pressure build up or strain weakening.

2.1 Use one or more of the screening or database methods to estimate the deformation. If the estimates of deformations are much less than what is tolerable; e.g. crest settlements are much less than the available freeboard before the earthquake for the seismic load being considered, allowing for the uncertainty of the method, accept that the likelihood or failure by overtopping is negligible for that loading.

2.2 If the deformations estimated by the screening or database methods are greater than this, use one or more of the simplified methods to estimate deformations. If the estimated deformations are significantly less than the available freeboard allowing for uncertainty in the method, for the seismic load being considered, accept that the likelihood or failure by overtopping is negligible for that loading.

2.3 Repeat this for the full range of seismic loading being considered.

2.4 If the deformations estimated by the simplified methods are greater than the available freeboard for the maximum seismic load being considered, either use risk based methods to assess whether remedial works to reduce deformations are required, or use an appropriate advanced numerical method to estimate

deformations, and then use risk based methods to assess whether remedial works are required.

(3) For embankments and foundations which are subject to liquefaction and/or to significant pore pressure build up or strain weakening.

3.1 Assess post earthquake factors of safety using the limit equilibrium method. If factors of safety for all reasonable lower bound estimates of the liquefied strength and allowing for pore pressure build up and strain weakening in other zones are greater than 1.1 accept that the likelihood of failure will be small and probably tolerable subject to the quality and extent of information available. This means that for mean estimates of liquefied strength the post earthquake factor of safety is likely to be > 1.3 or 1.5.

3.2 If factors of safety are lower than described above, estimate deformations using the simplified deformation analysis method allowing for the range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the very approximate nature of these estimates.

3.3 For high and extreme consequence dams, regardless of the results of (3.2) and, for other dams where factors of safety are lower than described in (3.2), carry out static numerical analyses of the post earthquake condition for a range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the approximate nature of these estimates.

3.4 For cases where the estimated likelihood of failure are intolerable, either design remedial works so the residual likelihoods of failure are tolerable, or use advanced numerical methods to make more refined estimates of deformations. Allow for the range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the still approximate nature of these estimates.

The extent to which an embankment is analysed for deformations should be consistent with the consequence category and size of the dam. In many cases it will be better to design and construct remedial measures than continuing to do more and more sophisticated analyses.

There may however be situations where the more sophisticated methods are warranted. Even for these the deformations are at best approximate, and controlled by the quality of the data input into the analyses, and the limitations of the methods of analysis.

The more sophisticated methods require expert input to the analysis and selection of properties and should not be carried out other than by experienced persons.

12.7.5 Screening methods

12.7.5.1 *USACE method*

Perlea and Beaty (2010) report that the USACE do not require deformation analyses for low to moderate height dams <60 m high if all the following criteria are met:

- Dam and foundation materials are dense, not subject to liquefaction, and do not include sensitive clays.

- The dam is well built and densely compacted to at least 95% of the laboratory maximum dry density or to a relative density greater than 80%.
- The slopes of the dam are 3:1 (H:V) or flatter and/or (*the slopes are steeper but*) the phreatic line is well below the downstream face of the embankment.
- The predicted peak horizontal ground acceleration (PGA) at the base of the embankment is no more than 0.20 g. Compacted clay embankments on rock or stiff clay foundations may offer additional resistance to deformations. Somewhat higher allowable PGA values may be justifiable for these dams on a case-by-case basis, although the PGA criterion should not exceed 0.35 g (USBR, 1989).
- The static factors of safety for all potential failure surfaces involving loss of crest elevation (other than shallow surficial slides) are greater than 1.5 under the loading and pore-pressure conditions expected immediately prior to the earthquake.
- The freeboard at the time of the earthquake is at least 3 to 5 percent of the dam height plus alluvial foundation, and not less than 0.9 m. Special attention should be given to the presence and suitability of filters for dams with modest freeboard.
- There are no appurtenant features related to the safety of the dam that would be harmed by small movements of the embankment.
- There have been no historic incidents at the dam that may indicate a limitation in its ability to survive an earthquake.

This approach may be used as a screening method. However for the SEE many dams will have a PGA greater than 0.2 g (or the above quoted upper limit of 0.35 g, as the case may be).

12.7.5.2 *Hynes-Griffin and Franklin (1984) pseudo-static seismic coefficient method*

Up until the 1970s, the pseudo-static analysis was the standard method of stability assessment for embankment dams under earthquake loading. The approach involved a conventional limit equilibrium stability analysis, incorporating a horizontal inertia force to represent the effects of earthquake loading. The inertia force was often expressed as a product of a seismic coefficient "k" and the weight of the sliding mass W. The larger the inertia force, the smaller the safety factor under the seismic conditions. In this approach a factor of safety (FOS) of <1 implies failure, whereas FOS >1 represents seismically safe conditions.

The seismic coefficients used in this approach were typically less than 0.2 and were related to the relative seismic activity in the areas to which they apply.

The pseudo-static method of analysis, despite its earlier popularity, was based on a number of restrictive assumptions. For instance, it assumed that the seismic coefficient acting on the potential unstable mass is permanent and in one direction only. In reality, earthquake accelerations are cyclic, with direction reversals. Also, the concept of failure used in the approach was influenced by that used in static problems where a factor of safety of less than one cannot be permitted, as the stresses producing this state will exist until large deformations change the geometry of the structure.

However, under seismic conditions, it may be possible to allow the FOS to drop below one, as this state exists only for a short time. During this time, earthquake-induced inertia forces cause the potentially unstable masses to move down the slope.

However before significant movement takes place, the direction of the earthquake loading is reversed and the movement of the soil masses stop as, once again, the FOS rises above one. In fact, experience shows that a slope is likely to remain stable post earthquake despite having a calculated pseudo static FOS less than 1.0.

Hynes-Griffin and Franklin (1984) refined this method to account for likely deformations based on Newmark sliding block analyses of 349 horizontal components of natural earthquakes and 6 synthetic records for a range of yield accelerations. From this permanent displacements u in cm were plotted versus N/A where N is the yield acceleration and A is the peak value of the earthquake acceleration at the base of the dam.

Hynes-Griffin and Franklin (1984) then used an arbitrary limit of 100 cm (one metre) displacement as representing tolerable displacements, and from Figure 12.44, used the upper bound plot of displacements, which gave $N/A = 0.17$. After allowing for amplification factor of 3 between the base acceleration and that at the crest of the dam, they concluded that a factor of safety of 1.0 with a seismic coefficient of one half of the peak ground acceleration at the base of the dam, would assure that deformations would not exceed 1 metre. Their suggested method is:

- Carry out a conventional pseudo static stability analysis using a seismic coefficient equal to one-half the predicted peak ground acceleration.
- Use a composite S-R strength envelope (Effective stress strength at low stresses, undrained strength at high stresses) for pervious soils and R undrained strength for clays, multiplying the strength in either case by 0.8.
- Use a minimum factor of safety of 1.0.

It should be noted that this method cannot be used in reverse. That is, just because the factor of safety is less than 1.0, it should not be assumed that the deformations will be greater than 1 metre because the upper bound deformation curve in Figure 12.44 has been used in the method.

In practice for most dams with static factors of safety about 1.5, the SEE pga will result in factors of safety using this method less than 1.0 so the method is not particularly useful. It is included here because the background to the method is often not appreciated by practitioners and to avoid its misuse.

12.7.6 Empirical database methods

Empirical database methods are based on recorded deformations, dam geometry and earthquake loading. The following summarizes the methods.

12.7.6.1 *Swaisgood (1998, 2003) empirical method for estimating crest settlements*

Swaisgood (1998, 2003) gathered data on crest settlement, dam height, type, depth of alluvium in the foundation, earthquake magnitude and peak ground acceleration and the focal distance of the dam to the earthquake.

Figure 12.45 shows relative crest settlement (settlement/{dam height + thickness of alluvium in the foundation}) versus peak ground acceleration (bedrock).

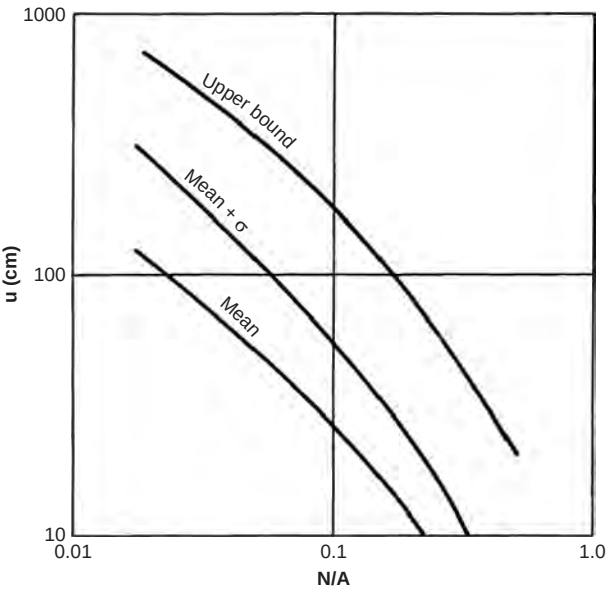


Figure 12.44 Permanent displacement u in centimetres versus N/A based on 354 accelerograms (Hynes-Griffin and Franklin, 1984).

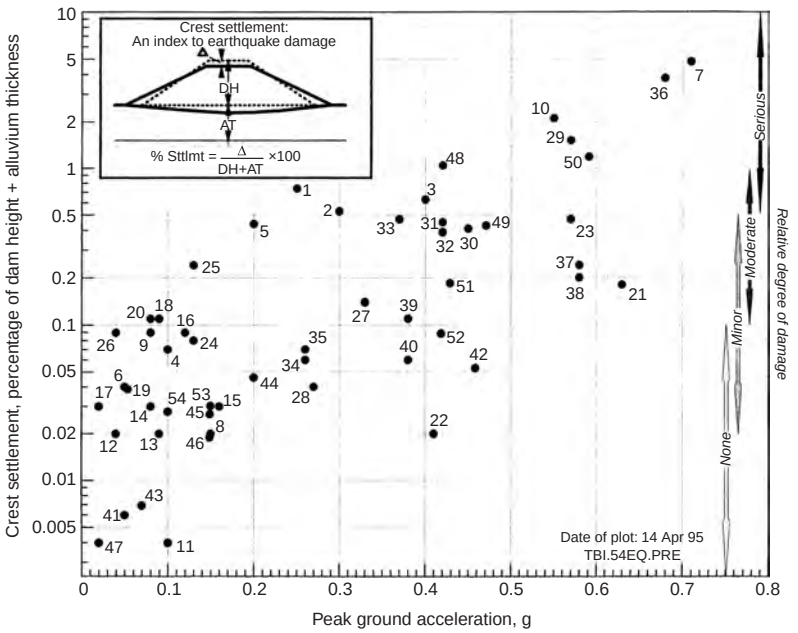


Figure 12.45 Relative crest settlement versus peak ground acceleration. E = Earthfill; ECRD = earth core rockfill; CFRD = concrete face rockfill; HF = hydraulic fill (Swaisgood, 1998, 2003).

Swaisgood (1998, 2003) recommended the following equations to predict settlement:

$$CS = SEF \times RF \tag{12.39}$$

where CS is the vertical crest settlement expressed as a percentage of the dam height plus the alluvium thickness. SEF is the seismic energy factor and RF is the resonance factor. These factors are calculated from:

$$SEF = e^{(0.72M+6.28PGA-9.1)} \tag{12.40}$$

in which M is the magnitude of the earthquake (M_L or local magnitude below 6.5, M_S magnitude of surface wave magnitude at 6.5 or above and PGA is the peak horizontal ground acceleration at the dam site as a fraction of the acceleration due to gravity.

$$\begin{aligned} RF &= 2.0D^{-0.35} \quad \text{for earthfill dams} \\ &= 8.0D^{-0.35} \quad \text{for hydraulic fill dams} \\ &= 0.12D^{-0.61} \quad \text{for rockfill embankments} \end{aligned} \tag{12.41}$$

in which D is the distance between seismic energy source and dam in kilometres.

This method should only be used to give an approximate estimate of settlements, where there is no potential for liquefaction or significant strain weakening. Account should be taken of the scatter in the data as shown in Figure 12.45 and in practice it is probably best just to use the figure rather than the equations.

12.7.6.2 *Pells and Fell empirical method for estimating settlement, damage and cracking*

Pells and Fell (2002, 2003) gathered data from 305 dams, 95 of which reported cracking, and classified these for damage according to the system shown in Table 12.7.

Figures 12.46 and 12.47 show plots of damage contours versus earthquake magnitude and peak ground acceleration for earthfill and earth and rockfill dams with

Table 12.7 Damage classification system for embankment dams under earthquake loading (Pells and Fell, 2002, 2003).

Damage class			
Number	*Description*	*Maximum longitudinal crack width(1) mm*	*Maximum relative crest settlement(2) (%)*
0	No or slight	<10 mm	<0.03
1	Minor	10–30	0.03–0.2
2	Moderate	30–80	0.2–0.5
3	Major	80–150	0.5–1.5
4	Severe	150–500	1.5–5
5	Collapse	>500	>5

(1) Maximum crack width is taken as the maximum width, in millimetres, of any longitudinal cracking that occurs.
(2) Maximum relative crest settlement is expressed as a percentage of the maximum dam height (from general foundation to rhe dam crest.

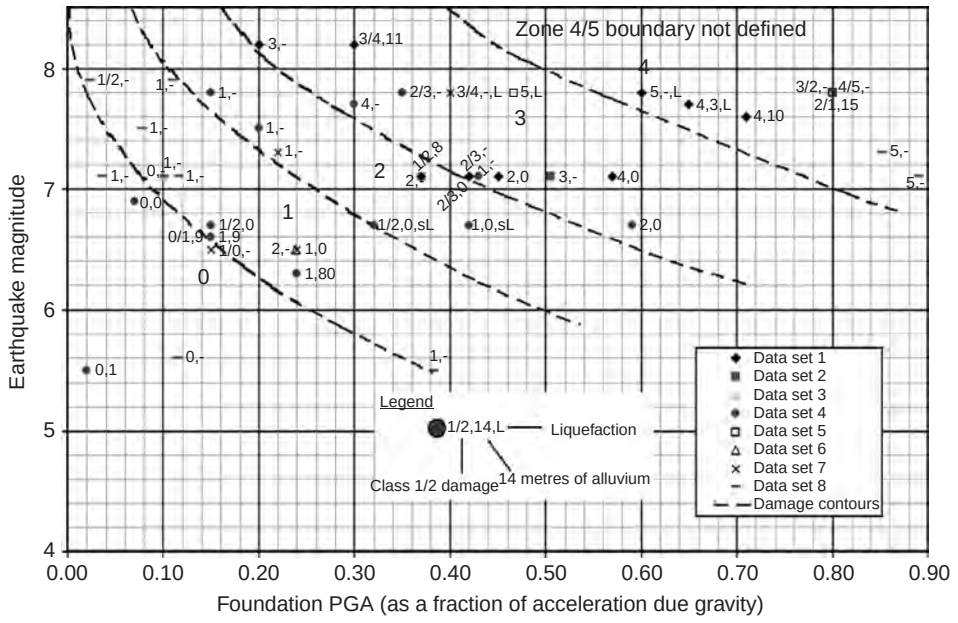


Figure 12.46 Earthfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration (as a fraction of acceleration due to gravity). Contours drawn without consideration for cases that had liquefaction (Pells and Fell, 2003).

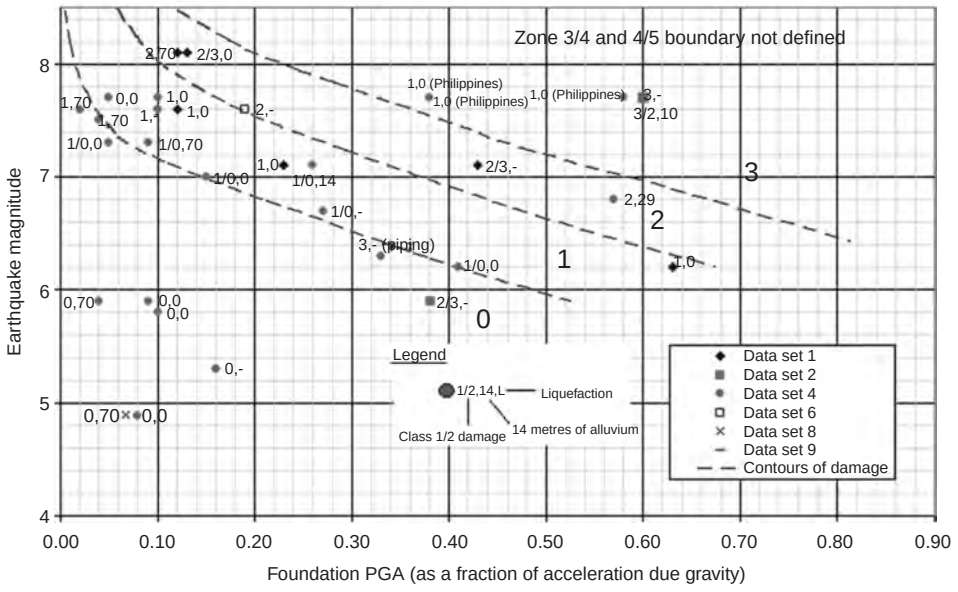


Figure 12.47 Earth and Rockfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration (as a fraction of acceleration due to gravity). Contours drawn without consideration for cases that had liquefaction. Boundaries shown assume rockfill is well compacted. For dumped or poorly compacted rockfill use Figure 12.46 to estimate the damage (Pells and Fell, 2003).

respect to crest settlement and cracking. These figures can be used to estimate the magnitude of crest settlement based on the range of relative crest settlements given for each damage class given in Table 12.7.

In terms of the potential for earthquake induced cracking, Pells and Fell (2002, 2003) concluded:

- Earthquakes cause settlement, lateral spreading and cracking of embankment dams. Slope instability may occur but it is not common. Longitudinal cracks are more common than transverse cracks and are mostly in the upper part of the dam, more likely on the upstream face than the downstream.
- There are a number of mechanisms that can lead to formation of either of these types of cracks. Many of these mechanisms are common to both seismic loading and normal operating loading. There are seen to be more mechanisms that may lead to the formation of longitudinal cracks that transverse cracks, largely because lateral displacement is more readily achieved in the upstream-downstream directions.
- For visible longitudinal cracks to occur the dam needs to experience a Magnitude 6.5 or greater earthquake, and a peak ground acceleration greater than about 0.15 g for earthfill dams and 0.3 g for earth and rockfill dams. Alternatively it needs an earthquake of Magnitude 7.0 or greater, and a PGA of 0.05 g for earthfill and, say, 0.15 g for earth and rockfill dams. For hydraulic fill dams, visible cracking may occur for Magnitude 6 earthquakes and 0.05 PGA.
- Dams which experience damage of Class 2 (relative settlement of 0.2% to 0.5% and/or longitudinal cracks 30–80 mm wide) or greater are highly likely to experience transverse cracking. However, transverse cracking has been observed at under M6.0 to M6.5 earthquakes, at PGA as low as 0.1 to 0.15 g.
- There is evidence to show that higher seismic loads will result in the formation of larger and deeper cracks and are more likely to cause transverse cracking due to the greater differential settlements across the valley.
- At low seismic loading, there is evidence to show that only one type of cracking is likely to develop. This may be either longitudinal (more common) or transverse, depending on factors within the embankment that make it predisposed to a particular form. It is the nature of the embankment, zoning and the foundation geometry and presence of compressible materials and not the seismic loading, that differentiate between which form of cracking develops and in particular determine whether transverse cracking occurs at low seismic intensities.
- The susceptibility of an embankment to a particular type of crack could not be related to dam type, but a weak relationship has been developed with dam shape – dams in steeper valleys tend to be more susceptible to transverse cracking near the abutments.

Fong and Bennett (1995) give some limited information on maximum transverse crack depth as a function of crest settlement and crest length. This shows:

$$\text{Maximum crack depth (m)} \approx c\ (\text{settlement/dam crest length})$$

where "c" is usually about 1000 to 2500, but ranges from 125 to about 10,000.

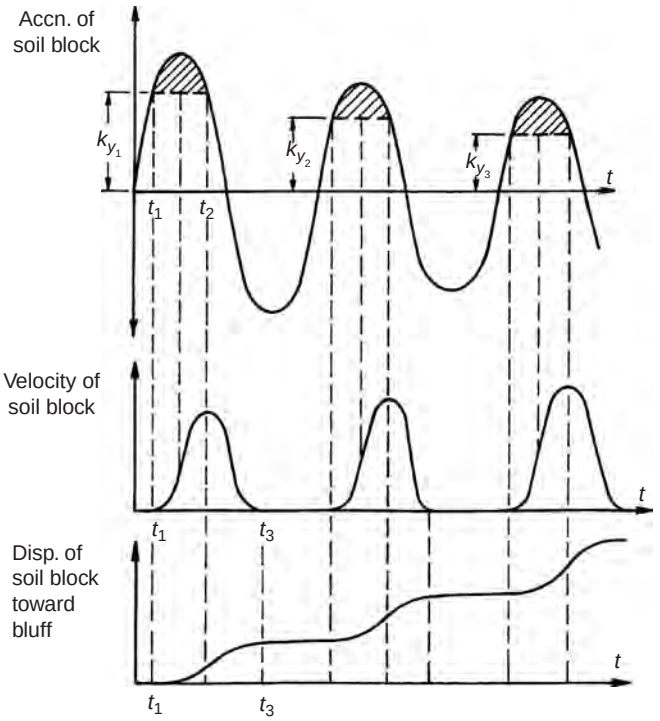


Figure 12.48 Double integration method for determination of the permanent deformation period of an embankment (Newmark, 1965).

Pells and Fell (2002, 2003) had limited data to indicate the ratio of transverse crack depth to crack width at the surface but what was available indicated that it is about 15 to 100, averaging about 40.

12.7.7 Simplified methods of deformation analysis for dams where liquefaction and significant strain weakening do not occur

12.7.7.1 *General principles*

These methods are all based on the Newmark (1965) approach. In this the sliding of a soil mass along a failure surface was likened to slipping of a block on an inclined plane. He envisaged that failure would initiate and movements would begin when the inertia forces exceed the yield resistance and that movements would stop when the inertia forces were reversed. Movement was assumed to happen only in the down-slope direction. He proposed that once the yield acceleration and the acceleration time history of a slipping mass are determined the permanent displacements can be calculated by double integrating the acceleration history above the yield acceleration as shown in Figure 12.48.

According to this approach, the permanent deformation in a sliding mass is a function of:

- The amplitude of the average acceleration time history of the sliding mass, which in turn is a function of the base motion, the amplifying factor of the embankment and the location of the sliding mass within the embankment.
- The duration of the earthquake, which is a function of the magnitude of the earthquake.
- The acceleration of the potential sliding mass.

The validity of the basic principles of Newmark's approach has been demonstrated by many investigators (e.g. Goodman and Seed, 1966; Ambraseys, 1973; Sarma, 1975 and Makdisi and Seed, 1978). It is generally found that, provided the yield acceleration is accurately evaluated, the approach can estimate the permanent displacement of a soil mass in reasonably good agreement with those observed during past earthquakes.

It is stressed that Newmark's approach is limited in application to compacted clayey embankments and dry or dense cohesionless soils that experience very little reduction in strength due to cyclic loading. The approach should not be applied where embankments or their foundations are susceptible to liquefaction or strain weakening because it will significantly underestimate displacements.

The following gives an outline of two of these methods. Readers will need to refer to the references given for the details.

12.7.7.2 *Makdisi and Seed (1978) method*

The Makdisi and Seed (1978) approach is based on Newmark's method, but modified to allow for the dynamic response of the embankment as proposed by Seed and Martin (1966). The approach was developed from a series of deformation analyses performed on a large number of embankments subjected to earthquake loading.

The approach has been widely used and accepted among practicing engineers.

Bray and Travasarou (2007) used this method as well as their own (described below) and for the dams they analysed the Makdisi and Seed (1978) method generally underestimated the displacements estimated using non-linear analyses. Perlea and Beaty (2010) also warn the method may not be appropriate for severe ground shaking because it does not model strength losses. They also point out it was based on few earthquake records.

12.7.7.3 *Bray and Travasarou (2007) method*

A simplified procedure was proposed by Bray and Travasarou (2007) for estimating earthquake induced permanent displacements in earth dams using a Newmark-type model. This procedure is based on the results from a set of simplified nonlinear analyses using 688 recorded ground motions from 41 earthquakes. The flexibility of the dam system and, the interaction between yielding and seismic loading, were considered by using a nonlinear coupled stick-slip deformable sliding model. The flexibility of the dam structure is captured through an estimate of the initial fundamental period Ts.

Key parameters of this procedure are the yield acceleration ky (in g), the initial fundamental period of the embankment, Ts, the value of spectral acceleration for a damping of 5% and a degraded response period equal to 1.5Ts.

Bray and Travasarou (2007) claim the method provides improved characterisation of the uncertainty involved in the estimate of seismic displacement. It can also be incorporated into a probabilistic framework.

12.7.8 Advanced numerical methods for estimating deformations during and post earthquake for non-liquefied and liquefied conditions

Dynamic numerical codes used in practice may be divided into two main categories: total stress codes, and effective stress codes. They are reviewed in Zienkiewicz *et al.* (1986), Finn (1993, 1988, 2000) Marcuson *et al.* (1992) and Perlea and Beaty (2010). A brief discussion of some of the more frequently used codes within each category is provided in the following sub-sections.

12.7.8.1 *Total stress codes*

The total stress codes, as can be inferred from the classification, are based on the total stress concept and do not take account of pore pressures in the analysis. Therefore they should only used in situations where the seismically induced pore pressures are negligible. The total stress codes may be divided into two main categories: (1) codes based on the equivalent linear (EQL) method of analysis, and (2) fully non-linear codes.

(a) Equivalent linear analysis (EQL)

The earlier total stress codes were based on the EQL method of analysis developed by H.B. Seed and his colleagues in 1972. EQL is essentially an elastic analysis and was developed for approximating non-linear behaviour of soils under cyclic loading. Typical of the EQL codes used in practice are: SHAKE (Schnabel *et al.*, 1972), QUAD-4 (Idriss *et al.*, 1973), QUAD4M (Hudson *et al.*, 1994) and FLUSH (Lysmer *et al.*, 1975). SHAKE is a one dimensional wave propagation program and is used primarily for site response analysis. QUAD-4. QUAD4M and FLUSH are two-dimensional versions of SHAKE and are used for seismic response analysis of dams and embankments.

Given the elastic nature of the EQL analysis, these codes cannot take account of material yielding and material degradation under cyclic loading. Therefore, they tend to predict a stronger response than actually occurs. Also, they cannot predict the permanent deformations directly. Indirect estimates of permanent deformations can however be obtained using the acceleration or stress data obtained from an EQL analysis and the semi-empirical methods proposed by Newmark (1965) and/or Seed *et al.* (1973).

(b) Fully non-linear analysis

More accurate and reliable predictions of permanent deformations can be obtained using the elasto-plastic nonlinear codes. Typical of the elasto-plastic non-linear codes used in the analysis of embankments are DIANA (Kawai, 1985), ANSYS (Swanson, 1992), FLAC (Cundull, 1993), etc. The constitutive models used in these codes vary from simple hysteretic non-linear models to more complex elasto-kinematic hardening

plasticity models. Compared to the EQL codes, the elasto-plastic non-linear codes are more complex and put heavier demand on computing time. However, they provide more realistic analyses of embankments under earthquake loading, especially under strong shaking. Critical assessments of non-linear elasto-plastic codes can be found in Marcuson *et al.*, (1992) and Finn (1993, 2000).

12.7.8.2 *Effective stress codes*

Effective stress codes allow modeling pore pressure generation and dissipation in materials susceptible to liquefaction and thus to obtain better estimates of permanent deformations under seismic loading. The effective stress codes may be divided into three main categories: fully coupled, semi-coupled and uncoupled.

(a) Fully coupled codes

In the fully coupled codes, the soil is treated as a two-phase medium, consisting of soil and water phases. Two types of pore pressures are considered, transient and residual. The transient pore pressures are related to recoverable (elastic) deformations and the residual pore pressures are related to non-recoverable (plastic) deformations. A major challenge in fully coupled codes is to predict residual pore pressures. The residual pore pressures, unlike the transient pore pressures, are persistent and cumulative and thus exert a major influence on the strength and stiffness of the soil skeleton. The transient pore pressures are cyclic in nature and their net effect within one loading cycle is often equal to zero. An accurate prediction of residual pore pressures requires an accurate prediction of plastic volumetric deformations. In the fully coupled codes this is often achieved by utilizing elasto-plastic models based on kinematic hardening theory of plasticity (utilizing multi-yield surfaces) or boundary surface theory with a hardening law.

Generally speaking, fully coupled prediction of pore pressures under cyclic loading is very complex and difficult. The validation studies performed on a number of these codes suggest that the quality of response predictions is strongly path dependent. When the loading paths are similar to the stress paths used in calibrating the models, the predictions are good. As the loading path deviates from the calibration path, the predictions become less reliable. Apart from the numerical difficulties, part of this unreliability is also due to the poor or less than satisfactory characterization of the soil properties required in the models. For instance, because of sampling problems, it is often very difficult to accurately determine volume change characteristics of loose sands as required by these models. In general, the accuracy of pore pressure predictions in fully coupled modes is highly dependent upon the quality of the input data.

Fully coupled codes include DNAFLOW (Prevost, 1981, 1988), DYNARD (Moriwaki *et al.*, 1988; PLAXIS (2012); ABAQUS (2012) and FLAC (Itasca, 2011)

As described in Perlea and Beaty (2010), FLAC contains a number of general purpose constitutive models but also provides for the use of user-defined constitutive models. These include a bounding surface hypo-plasticity model Wang and Makdesi (1999) by AMEC Geomatrix Inc.; FLAC-UBCSAND; (Beaty and Byrne, 2011) and PM4S and (Dafalais and Manzari, 2004; Boulanger, 2010; Boulanger and Ziotpoulou, 2012; Ziotopoulou and Boulanger, 2012). PLAXIS and ABAQUS also offer user defined constitutive models such as UBCSAND.

(b) Semi-coupled codes

Compared to the fully coupled codes, the semi-coupled codes are more robust and less susceptible to numerical difficulties. However they are theoretically less rigorous. In these codes empirical relationships, such as those proposed by Martin *et al.* (1975) and Seed (1983) are used to relate cyclic shear strains/stresses to pore pressures. The empirical nature of the pore pressure generation in these codes generally puts less restriction on the type of plasticity models used in the codes. The parameters they require are often routinely obtained in the laboratory or in the field.

Semi coupled codes include DESRA-2 (Lee and Finn, 1978), DSAGE (Roth, 1985), TARA-3 (Finn *et al.*, 1986). FLAC (Cundall, 1993, Itasca, 2011) can also be used in this mode as in the URS cycle weighted model described in Perlea and Beaty (2010).

12.7.8.3 *Summary*

More advanced dynamic methods are potentially expensive and should only be done by very experienced persons who understand the limitations of the analysis and the need to use well considered properties and good understanding of the geotechnical model and mechanisms. The analysis methods are controlled by the quality of data put into them, the limitations of the methods themselves and particularly of those doing the analysis.

They are in most cases only to be contemplated for dams and foundations subject to liquefaction and where the extra refinement of the analyses are warranted, e.g. where the simplified methods are not really applicable and/or are resulting in marginal assessments on whether remedial works are required.

Any non-linear deformation analysis needs to be documented in sufficient detail that it can be reasonably scrutinized. All input parameters, all numerical details (boundary conditions, damping parameters etc.) and enough output plots to evaluate reasonableness of initial conditions and dynamic responses would be required. These should be reviewed by persons expert in the practice.

Perlea and Beaty (2010) give an overview of USACE practice and examples of use of many of the programs described above. Stark *et al.* (2012), Friesen and Balakrishnan (2012) give useful examples.

12.8 DEFENSIVE DESIGN PRINCIPLES FOR EMBANKMENT DAMS

The concept of "defensive design" of embankment dams for earthquake was developed by Sherard (1967) and Seed (1979a) and endorsed by Finn (1993), ICOLD (1986c, 1999, 2001).

The general philosophy is to apply logical, commonsense measures to the design of the dam, to take account of the cracking, settlement and displacements which may occur as the result of an earthquake. These measures are at least as important (probably more so) as attempting to calculate accurately the stability during earthquake or the likely deformations. The most important measures which can be taken are:

(a) Provide ample freeboard, above normal operating levels, to allow for settlement or slumping or fault movements which displace the crest.
(b) Use well designed and constructed filters downstream of the earthfill core (and correctly graded rockfill zones downstream of a concrete face for concrete face

rockfill dams) to control erosion of the core (or face), should such occur in the earthquake. Filters should be taken up to the dam crest level, so they will be effective in the event of large crest settlements, which are likely to be associated with transverse cracking. For larger dams, full width filters (2.5 m to 3 m) might be adopted instead of narrower (say 1.5 m) filters placed by spreader boxes. This would give greater security in the event of large crest deformations.

(c) Provide ample drainage zones to allow for discharge of flow through possible cracks in the core. For example ensure that at least part of the downstream zone is free draining or that extra discharge capacity is provided in the vertical and horizontal drains for an earthfill dam with such drains. In this regard some embankment dam types are inherently more earthquake resistant than others. In general the following would be in order of decreasing resistance:

- Concrete face rockfill,
- Sloping upstream core earth and rockfill,
- Central core earth and rockfill,
- Earthfill with chimney and horizontal drains,
- Zoned earth-earth rockfill,
- Homogeneous earthfill,
- Upstream construction tailings and other hydraulic fill.

(d) Avoid, densify, drain (to be non-saturated) or remove potentially liquefiable materials in the foundation or in the embankment. Figure 12.49 shows the

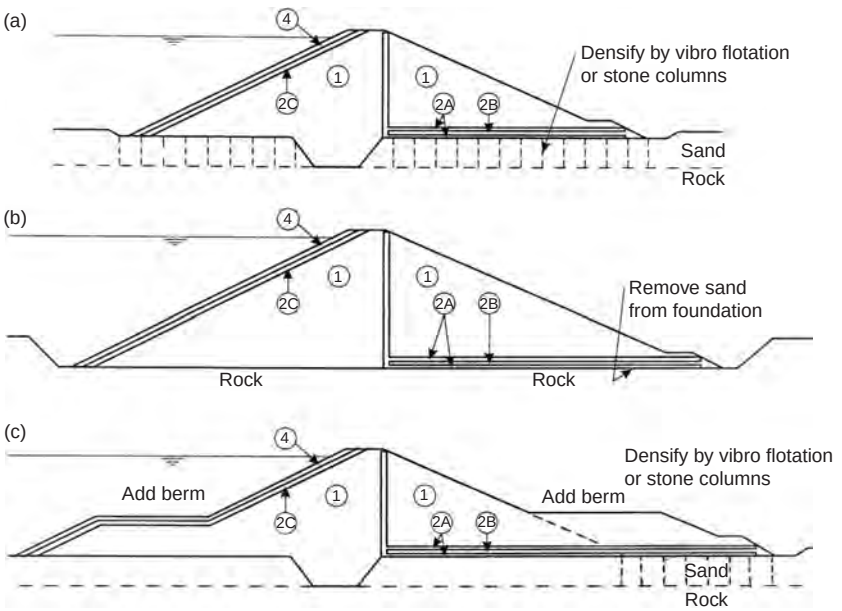


Figure 12.49 Principles of ground improvement or treatment to reduce the likelihood of liquefaction in dam foundations. (a) Densification by vibro-flotation or stone columns. (b) Remove potentially liquefiable soils. (c) Densification by vibro-flotation or stone columns under a berm for an existing dam.

principles for new dams and for remediation of existing dams. Filters, rockfill and other granular materials in the embankment should be well compacted if they are likely to become saturated, so they will be dilatant and not liquefy or build up pore pressure under cyclic loading.

(e) Avoid founding the dam on strain weakening clay soils, completely weathered rock or weak rock with the potential to strain weaken. In many older, smaller dams, this was not done. Post earthquake stability can be an issue if the earthquake causes even relatively minor movements which can take the foundation strength from peak to a strength close to or at residual.

(f) The foundation under the core should so far as is practicable be shaped to avoid sharp changes in profile across the valley. These are likely to make the core more susceptible to cracking due to differential settlement under earthquake (and normal) loading. Details of foundation slope modification are given in Section 17.6. ICOLD (1999) also suggest that it is desirable to keep the slope in the upstream-downstream direction across the core horizontal, or sloping gently upstream, in the upper 30 m of the dam. This is to assure watertight contacts after shaking and/or to preclude excessive settlement of the embankment materials. In practice it is seldom practicable to achieve much by slope modification and the potential effects of cracking have to be managed by providing filters and drainage capacity as described above.

There are a number of other less important measures which are listed by Sherard (1967), Seed (1979a) and Finn (1993). These include:

(g) Use a well-graded filter zone upstream of the core to act as a crack stopper, possibly only to be applied in the upper part of the dam. The concept is that, in the event that major cracking of the core occurs in an earthquake, this filter material will wash into the cracks, limiting flow and preventing enlargement of the crack. If well-designed filters are provided downstream, this upstream filter is of secondary importance.

(h) Provide crest details which will minimize the potential damage. The common practice of using steeper slopes near the crest (usually to provide for camber of the crest) and the use of wave walls in concrete face rockfill dams will make the crest more susceptible to damage.

(i) Flare the embankment core at abutment contacts, where cracking can be expected, in order to provide longer seepage paths. More important is to consider the detailing of the contact with concrete walls and the provision of filters downstream of the contacts. This detailing is discussed in Section 13.6.

(j) Locate the core to minimize the degree of saturation of materials on the downstream side of the section (e.g. use sloping upstream core). Finn (1993) also suggests positioning chimney drains near the central section of the embankment). These measures are intended to reduce to a minimum the extent of saturated zones which are more likely to reduce strength on cyclic loading. They are particularly relevant where sand, silty sand and sand-gravel soils are used in the dams as these are most susceptible to liquefaction. If all materials in the dam are well compacted this requirement is not important.

(k) Stabilize slopes around the reservoir rim (and appurtenant structures such as spillways) to prevent slides and rock falls into the reservoir or onto the structures.
(l) Provide special details if there is likelihood of movement along faults or seams in the foundation.
(m) Site the dam on a rock foundation rather than soil foundation (particularly if it is potentially liquefiable) where the option is available.
(n) Use well graded (densely compacted) sand/gravel/fines or highly plastic clay for the core, rather than clay of low plasticity (if the option is available) (Sherard, 1967).

Seed *et al.* (1985) suggest that the slopes of concrete face rockfill dams should be flattened to limit displacements in earthquake. The authors are not convinced this is necessary for well-compacted rockfill.

When assessing an existing dam, the use of these "defensive design" measures is seldom practical (except in remedial works). However, it is useful to gauge the degree of security the existing dam presents by comparing it with this list. Where the dam fails to meet many or most of these features, particularly (a) to (e), this may be a better guide to the fact that the dam may not be very secure against earthquake than a lot of analysis.

Dams which have well designed and constructed filters, have adequate stability against normal loads, and do not have liquefiable or strain weakening zones or foundations, will be able to withstand the loading from very large earthquakes.

12.9 METHODS FOR UPGRADING EMBANKMENT DAMS FOR SEISMIC LOADS

12.9.1 General approaches

The method or methods suitable for upgrading a dam and its foundation for seismic loads will be site specific and will require a thorough understanding of the liquefaction mechanics, where liquefaction is potentially of concern, the extent of liquefied foundation and embankment, and the consequences for the dam. It will also depend on the objectives, probably measured in residual risk terms.

The remedial measures may consist of one or more of the following:

1. Do nothing and accept the potential damage and risks.
2. Adding filters and possibly raising the crest level of the embankment so the consequences of deformations and the resulting risks are reduced to tolerable levels.
3. Modifying the liquefiable soil in the foundation (and the embankment if applicable), and/or constructing stabilizing berms to limit deformations to tolerable levels.

It is not uncommon to use a combination of methods, e.g. to carry out treatment on the downstream but do nothing upstream and tolerate the risks posed by upstream deformations.

12.9.2 Upgrading of embankment dams not subject to liquefaction

The following are the most common remedial measures which may be required:

1. Provision of filters or upgrade of existing filters in the upper part of the dam. This is usually done to reduce risks of internal erosion and piping to tolerable levels for flood and normal loading, and the upgrade for earthquake loads is achieved at the same time.
2. For the few dams where the existing freeboard is insufficient to cope with earthquake induced deformations, the embankment may have to be raised, often along with raising for increased flood capacity.
3. Where there are strain weakening soils such as high clay content high plasticity over-consolidated clays in which strains may localise under earthquake deformations, it may be necessary to add a stabilizing berm.

12.9.3 Embankment dams subject to liquefaction

The following are the most common remedial measures which may be required:

(a) Construction of a stabilizing berm, most commonly founded upon the potentially liquefiable soil after it has been treated by ground improvement as shown in Figure 12.49(c). Alternatively remove the potentially liquefiable soil from the foundation of the berm.
(b) The purpose of the ground improvement is to either densify the soil so it will no longer liquefy and/or to provide additional strength to the potentially liquefiable soil strata. Stone columns and other vertical drainage measures have the added role of draining excess pore pressures developed by the cyclic loading of the earthquake so reducing the likelihood of liquefaction.
(c) Provision of filters or upgrade of existing filters in the upper part of the dam to reduce the risks of internal erosion and piping to tolerable levels.
(d) Where freeboard is insufficient, the embankment may have to be raised.

These works may be required only on the downstream of the embankment, or on both upstream and downstream. Quite commonly only part of the length of the embankment may require remedial works; e.g. if only part is founded upon liquefiable soil.

The ground improvement method used will depend on the nature and depth of the liquefiable soils, the techniques available and cost. Mitchell (2008), Idriss and Boulanger (2008) and Seed *et al.* (2003) discuss the various ground improvement methods and their limitations. These are summarized in Table 12.8.

The most widely used methods in the authors experience have been "stone" columns, deep soil mixing, and removal and replacement. Examples are described in Davidson *et al.* (2003), Toose *et al.* (2007), Mejia (2005), Dise *et al.* (2004) and Luehring *et al.* (2001).

Table 12.8 Summary of ground improvement methods and their application for remedial works for liquefiable foundations.

Method	*Method of Improvement*	*Soils for which it the method is applicable*	*Limitations and Comments*
Vibroflotation	Densification, increased lateral stresses	Sand, sand with some silt, gravelly sand	Ineffective if silt content >20% approx. May not penetrate gravelly strata.
Vibro-replacement e.g. stone columns	Densification increased lateral stresses, reinforcement, increased drainage	Sand, silt, clay	Ineffective densification if silt content >20% approx. May not penetrate gravelly strata. Columns may be constructed by driving casing, removing soil from within casing, and compacting "stone" as casing is withdrawn. Treatment must penetrate through liquefiable soil or treatment will be ineffective at the base.
Dynamic compaction	Densification increased lateral stresses	Sand, silty sand	Ineffective if silt content >20% approx. Effective only in the upper 10 m. Quality control difficult. Vibration of adjacent structures.
Compaction grouting	Densification increased lateral stresses, reinforcement	Sand, silt, clay	Ineffective at depths <6 m. Augers must penetrate through liquefiable soil or treatment will be ineffective at the base. Quality control may be difficult.
Deep soil mixing	Reinforcement, reduce earthquake induced strains and pore pressures	Sand, silt, clay	Augers must penetrate through liquefiable soil or treatment will be ineffective at the base. Quality control may be difficult. Soil-cement elements may be brittle.
Jet grouting	Reinforcement, reduce earthquake induced strains and pore pressures	Sand, silt, clay	Uses very high pressures so should not be used where hydraulic fracture may damage the embankment core. Difficult to control diameter of treatment.
Vertical drainage	Drainage	Sand, sand with some silt	Ineffective in silty sands and silts.
Permeation grouting	Reinforcement	Sand, gravel, sandy gravel, gravelly sand	Particulate grouts e.g. microfine cement will not penetrate fine sands because the sand acts as a filter. Chemical grouts expensive and some toxic. Quality control difficult.
Removal and replacement	Removes liquefiable soil	All soils	Dewatering, stability during construction may be problems. Gives high degree of confidence in final product.

There are detailed design matters which need to be considered when designing "stone" columns. These include:

(1) If the columns are situated on the downstream side and there is no cutoff through the alluvium upon which the dam is constructed, the columns may acts as drains for seepage water. In this case they should be backfilled with sandy gravel designed to act as a filter to the surrounding soil or backward erosion may initiate into the columns. This can create problems with the construction of the columns as the backfill is low permeability. For some projects the backfill has been designed as a "some erosion" filter, using the Foster and Fell (2001) method.
(2) The composite strength of the columns and the surrounding soil is estimated allowing for the relative areas of the columns and the surrounding soil. The strength of the surrounding soil should be taken as the liquefied strength unless it can be demonstrated that the soil between the columns is or densified so that it is no longer liquefiable or the soil is sufficiently permeable to allow dissipation of pore pressures built up during the cyclic loading. The latter can be assessed by the method of Seed and Booker (1977). Idriss and Boulanger (2008) indicate that method has been refined by Onoue (1988), Iai and Koizumi (1986) and Pestana *et al*. (2000), but they caution that uncertainties in the hydraulic conductivities of the surrounding soil (particularly as it is disturbed by construction of the columns) and the columns themselves make it difficult to have confidence of efficient drainage except in relatively high permeability soils.
(3) Seed *et al*. (2003) discuss the effect of the stiffer stone columns attracting the cyclic shear stresses and potentially reducing the build up of pore pressure in the liquefiable soil. They caution against relying upon this because the columns still flex if they are longer than three times their diameter.

Chapter 13

Embankment dam details

13.1 FREEBOARD

13.1.1 Definitions and overall requirements

a) *Freeboard*. The freeboard for a dam is the vertical distance between a specified reservoir water surface level and the crest of the dam, without allowance for camber of the crest of the dam.
b) *Normal freeboard* is the vertical distance between the crest of the dam without allowance for camber, and the normal reservoir full supply level.
c) *Minimum freeboard* is the vertical distance between the crest of the dam without allowance for camber, and the maximum reservoir water surface (MRWS) that results from routing the inflow design flood (IDF) through the reservoir.

The objective of having freeboard is to provide assurance against overtopping resulting from wind set up and wave run up.

Other factors which may influence the selection of freeboard include:

- Reliability of design flood estimates.
- Spillway gate reliability and the effect of a gate or gates failing to operate during the flood.
- Assumptions made in flood routing.
- Potential changes in design flood estimates, either through changes in flood estimation techniques or due to changed catchment conditions.

In some cases there may be a need to consider waves which could be generated by landslides failing into the reservoir and waves or seiches caused by earthquake faults in the reservoir area.

It should be noted that camber is not part of the freeboard, since it is provided to allow for the long term settlement of the dam.

13.1.2 Examples of freeboard requirements

13.1.2.1 *New embankment dams*

USBR (1992, 2012a) have the following freeboard requirements for new embankment dams.

Freeboard at maximum reservoir water surface elevation during IDF

(1) The minimum freeboard should be the greater of (a) 0.9 m or (b) the sum of the wind set up and wave run up that would be generated by the typical winds that would be expected to occur during large floods, as determined after seeking advice from local authorities and meteorologists. They point out that for large reservoirs and catchments the wind may be independent of the storm event that created the flood, and suggest a wind with a 10% exceedance probability (1 in 10 years) should be used.

The first criterion is USBR historic standard for embankment dams. To some degree, it accounts for uncertainty in wind loadings as well as possible changes in both wind and hydrologic loadings through the life of the dam. They indicate that the standard of 0.9 m should be increased when either there is significant uncertainty in these loadings or other freeboard factors, or when the dam poses an unusually high risk of failure due to overtopping.

If winds that occur during the IDF when the reservoir is at the MRWS are not independent of the hydrologic event (for example, potentially with a hurricane-related storm event or when a flood occurs over a small watershed), meteorologists should be consulted to estimate the wind velocities that would occur at this time.

(2) *Normal water surface freeboard* – the freeboard above normal reservoir surface (MRWS), e.g. full supply level, should be the wind set up and wave run up for the highest sustained wind velocity that could reasonably occur at the site. The USBR standard for new embankment dams is to apply a wind velocity of 100 miles per hour (160 km/hr) over the MRWS to derive the required normal freeboard. This velocity is not necessarily applicable elsewhere.

The elevation on the dam considered to be the "normal" reservoir water surface is the highest within the typical range of annual operations (not including flood operations).

(3) *Intermediate water surface freeboard* – the intermediate freeboard requirement should be determined so that it has an acceptably remote probability of being exceeded by any combination of wind generated waves, wind set up and reservoir surfaces occurring simultaneously. This is a particularly important consideration if reservoir water levels are expected to exceed the MRWS frequently or are often near the MRWS and maintain the high levels for an extended period of time. It is in effect a risk based requirement.

The adopted embankment crest level should satisfy all three requirements.
The design wind speeds are discussed further in Section 13.1.4.2.

13.1.2.2 *Existing embankment dams*

For existing embankment dams USBR (1992, 2012a) require that, if the maximum water surface elevation is so close to the dam crest that wind generated waves and setup would wash over, or if the maximum water surface elevation is higher than the existing crest, the potential of this overtopping to cause failure of the embankment should be assessed.

This will be dependent upon the following factors:

- *Depth, Velocity, and Duration of Overtopping* – The deeper, faster, and longer that water flows over a dam, the more likely the possibility of failure.
- *Crest Elevations, Width, and Slope* – When overtopping is a potential concern, low spots concentrate flow and thus are far more likely to lead to erosion as compared to a dam that has a uniform elevation and sheet flow during overtopping.
- *Crest and Downstream Slope Face Materials* – Dams with crests paved with bitumen or concrete would be able to withstand overtopping somewhat better than dams with minimal crest protection. Likewise, dams with erosion-resistant covering on the downstream face, especially near the top of the slope, where overtopping flows would accelerate rapidly, so potentially initiating head-cutting of the crest, should be expected to take a small amount of overtopping.

While it is common to assume that an embankment dam would fail as a result of any amount of overtopping, there are many examples of dams surviving some overtopping.

USBR (2012a) suggest that a risk analysis may be used to estimate the probability of dam failure and consequences of failure by overtopping.

This may be assessed by developing "fragility curves" to estimate the probability of failure given overtopping. These typically relate the probability of failure to the depth of overtopping and are based on published case data. Figure 13.1 shows an example. This should not be used for other dams as the data is site specific.

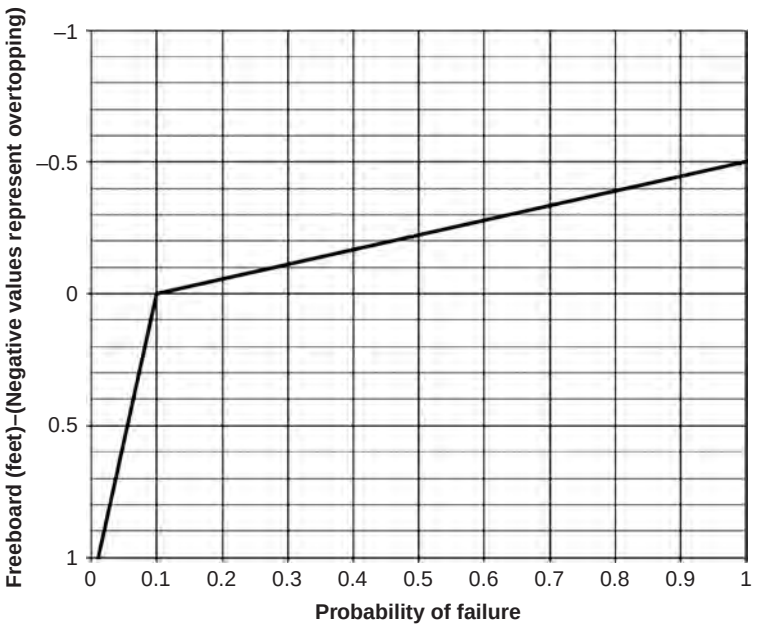


Figure 13.1 Example of a fragility curve relating the probability of failure to the depth of overtopping (USBR, 2012).

Table 13.1 Freeboard requirements for design of small dams and for preliminary studies (USBR, 1987, 2012a).

Largest fetch (km)	*Normal freeboard (m)*	*Minimum freeboard (m)*
Less than 1.6	1.2	0.9
1.6	1.5	1.2
4	1.8	1.5
8	2.4	1.8
16	3.0	2.1

The Bureau of Reclamation Best Practices and Risk Methodology manual (USBR 2012b) gives guidance on how to assess fragility curves for flood overtopping of embankment dams and levees. This is largely based on judgemental probability assessments that are supported by engineering assessments of the hydraulics of overtopping flows and erosion resistance of surface materials (such as vegetated slopes or rockfill slopes). Additional information on the engineering assessments of overtopping of embankment dams and levees due to continuous overtopping or overtopping by waves can be obtained from FEMA (2007), EurOtop (2007), Frizell *et al.* (1998) and Temple and Irwin (2006).

13.1.2.3 *Suggested approach for determining freeboard*

It is suggested that the USBR approach be adopted in principle. The design wind velocities should be determined in consultation with meteorologists and to satisfy National Guidelines for the country in which the dam is located. The consequences category of the dam would also need to be considered.

13.1.3 Estimation of wave run up freeboard for design of small dams and for feasibility and preliminary design

For feasibility and preliminary design studies and, for small dams, it is suggested that the method outlined in USBR (1987, 2012a) be adopted. This is based on a wind velocity of 160 km/hr (100 miles per hour) for determination of normal freeboard and 80 km/hr (50 miles per hour) for minimum freeboard. The effect of wind setup is ignored.

For rip-rapped slopes the freeboard requirements are as tabulated in Table 13.1. For dams with a smooth pavement or soil cement upstream slope, depending on the smoothness of the surface, freeboard of up to 1.5 times those shown in Table 13.1 should be used.

13.1.4 Estimation of wind setup and wave run-up for detailed design

The following outlines the method described in USBR (2012a). This is based on USACE (2002, 2008, 2011) and supersedes the methods described in USBR (1992) and Fell *et al.* (2005).

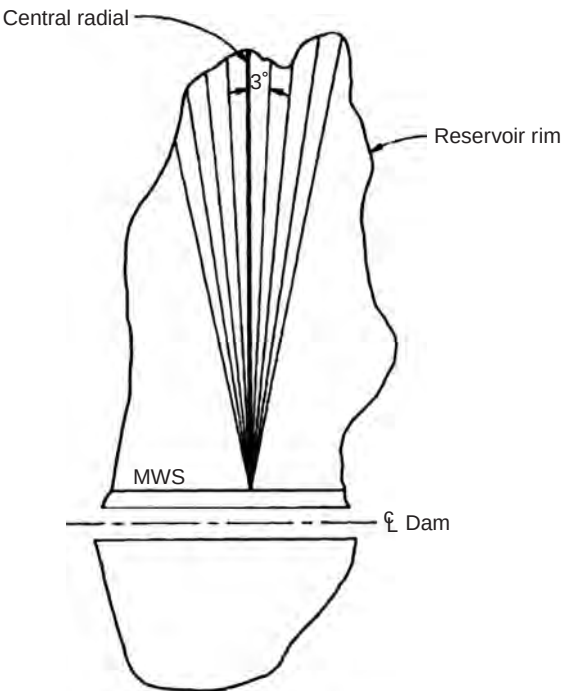


Figure 13.2 Method for calculating effective fetch (USBR, 2012).

The procedures are too lengthy to include here other than in summary form and for detailed design of important structures readers should refer to the original texts (USBR 2012a, USACE 2002, 2008, 2011).

13.1.4.1 *Fetch*

In reservoirs, fetches are limited by the land surrounding the body of water. The shorelines are irregular and an effective fetch is calculated by constructing nine radials from the point of interest on the dam, at 3 degree intervals as shown in Figure 13.2. The length of each radial is measured and arithmetically averaged as in Equation 13.1.

This calculation should be performed for several directions where the central radial is normal to the dam axis and also the direction where the total; spread results in the longest possible set of radials. For each fetch calculated the angle of the central radial in respect to the dam axis should be determined. This will be used to adjust the wave run-up.

$$\text{Fetch} = \sum_{1}^{n} (\text{length of radials})/(\text{number of radials}) \tag{13.1}$$

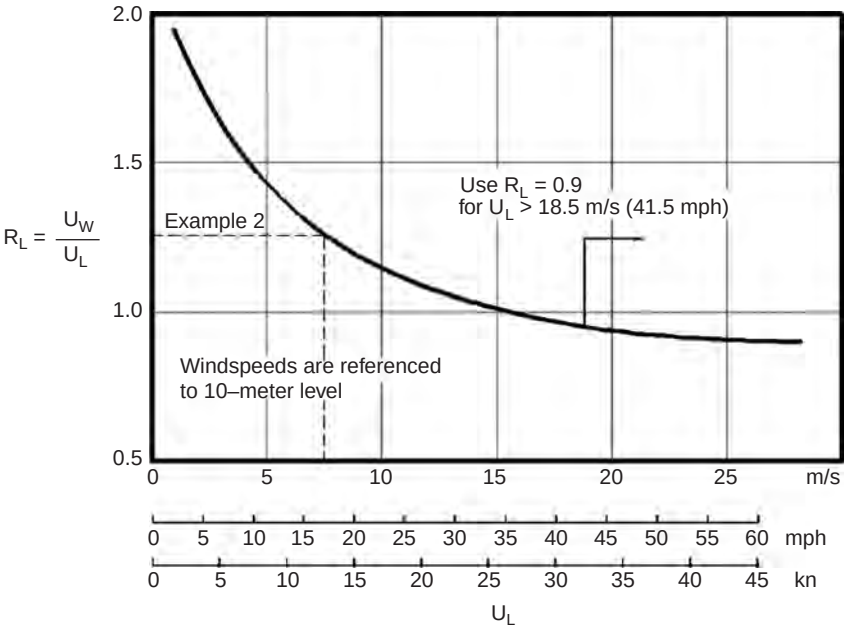

2.0
1.5
1.0
0.5
$R_L = \frac{U_W}{U_L}$
Example 2
Use R_L = 0.9
for U_L > 18.5 m/s (41.5 mph)
Windspeeds are referenced
to 10–meter level
0 5 10 15 20 25 m/s
0 5 10 15 20 25 30 35 40 45 50 55 60 mph
0 5 10 15 20 25 30 35 40 45 kn
U_L

Figure 13.3 Ratio of wind velocity over water to wind velocity over land as a function of wind velocity over land (USBR, 2012; USACE, 2008).

13.1.4.2 *Design wind*

Design wind velocity estimates should be obtained from the Bureau of Meteorology or equivalent organisation. This may be done for a range of wind durations and annual likelihood of occurrence.

The critical wind duration depends on the fetch; longer durations are critical for longer fetches. The estimates should allow for local topographic effects and if known the likely wind direction for high wind velocities.

Generally design velocities are controlled by the fetch. The duration needed for a given wind velocity to generate the highest waves for a given fetch is designated the minimum duration which can be estimated from (USBR, 2012a):

$$t_{min} = 1.6[(F)^{0.67}/(V_W)^{0.34}] \tag{13.2}$$

where
t_{min} = Minimum wind duration to generate maximum wave height in hours
F = Fetch in km
V_W = Wind velocity over water in km/hr

The estimates should be adjusted from the over land wind velocity to over water wind velocity by applying the correction factors shown in Figure 13.3. In Figure 13.3 U_L is the wind velocity over land and U_W is the wind velocity over water.

Table 13.2 Wave height reduction factor R_H (adapted from USBR, 2012, USACE 2008).

Fetch angle β degrees	*Wave height reduction factor R_H*
0	1.0
20	0.96
40	0.90
60	0.84
80	0.75

USBR (2012a) describes how these data may be used to estimate the annual probability of over water wind velocities being exceeded.

In some cases, it may be reasonable to consider the predominant wind directions relative to the fetch direction when deciding upon the appropriate wind velocity to use for the determination of the required freeboard. Factors to think about here would include the size of the storage and the expected timing of the peak water level during a flood. In small storages, where thunder storms would be the likely reason for the inflow flood and the peak water level would be reached very quickly, the critical wind should be assumed as coming from any direction.

13.1.4.3 *Wave height*

For the estimation of the minimum freeboard the significant wave height in metres (Hs), which is the average of the highest one-third of the waves in the wave spectrum, should be used. Hs can be estimated from Equation 13.3.

$$Hs = 0.00366(F)^{0.5}V_W(1.1 + 0.01V_W)^{0.5} \tag{13.3}$$

where
Hs = Significant wave height in metres
F = Fetch in km
V_W = Wind velocity over water in km/hr

This equation is valid for most reservoirs where the reservoir is relatively deep compared to the wind generated wave length (i.e. water depth >0.5L, where L is the deep water wave length calculated in Equation 13.4).

For the normal freeboard computation, the run up should be calculated using the average of the highest 10% of waves, which is 1.27Hs. A wave height of 1.67Hs should be used if overtopping by only an infrequent wave is permissible. This is the wave height exceeded by only 1% of the waves.

Wave heights for fetches that are not normal to the dam axis should be reduced by the factors shown in Table 13.2. In this table β = the angle between the centre radial of the fetch and a line normal to the dam axis. A value of $\beta = 0$ is for a fetch normal to the dam axis.

For shallow reservoirs it is suggested that US Corps of Engineers (2002, 2011) Part VI be used.

13.1.4.4 *Wave length and wave period*

The deep water wave length L (in metres) can be computed from Equation 13.4:

$$L = 1.56T^2 \tag{13.4}$$

where T is the wave period determined from:

$$T = 0.335F^{0.33}V_W^{0.33}(1.1 + 0.01V_W)^{0.167} \tag{13.5}$$

13.1.4.5 *Wave run-up*

If a deep water wave reaches a sloping embankment without major modification in characteristics, the wave will ultimately break on the embankment and run up the slope to a height governed by the angle of the slope, the roughness and permeability of the embankment surface, and the wave characteristics. Wave run-up, R, is the vertical difference between the maximum level attained by the rush of water up the slope and the still water elevation. To compute the run-up, a surf similarity factor for peak wave heights, ξ_p is first computed from the following equation:

$$\xi_p = 1.25T(\tan\alpha)/(Hs)^{0.5} \tag{13.6}$$

where
α = slope angle of the upstream slope from the horizontal (degrees)
Hs = significant wave height (m)

The run-up R is calculated from:

$$R = Hs(A\xi_p + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \tag{13.7}$$

where
R = Run-up on a relatively impermeable slope (e.g. the upstream slope of an embankment dam) (m)
Hs = Significant wave height (m)
ξ_p = Surf similarity factor (from the previous equations)

A, C = Coefficients dependent on ξ_p (see Table 13.4) and the probability of the run-up (2 percent is used for freeboard and riprap calculations)

$\gamma_r\gamma_b\gamma_h\gamma_\beta$ = Reduction factors derived as follows:

γ_r is a reduction factor to account for the roughness of the slope taken from Table 13.3. For riprap, a value of 0.55 is suggested.

γ_b is a reduction factor for the influence of a berm ($\gamma_b = 1.0$ for upstream slopes without a berm). For upstream slopes with a berm refer to USACE (2002, 2011).

γ_h is a reduction factor for the influence of shallow-water conditions, where the wave height distribution deviates from the Rayleigh distribution ($\gamma_h = 1.0$ for Rayleigh distributed waves).

γ_β is a reduction factor to account for the direction of the fetch relative to the dam axis and is determined from Figure 13.4.

Given the surf similarity factor for peak waves, ξ_p, Table 13.4 is used to derive the variables A and C . These values are for the average of the highest 2 percent of the run-ups which is commonly used for freeboard and riprap analysis.

Table 13.3 Surface roughness reduction factor (valid for $1 < \xi_p < 3$–4) [16b].

Type of slope surface	γ_r
Smooth, concrete, asphalt	1.0
Smooth block revetment	1.0
Grass (3 cm in length)	0.90–1.0
One layer of rock, diameter D, (Hs/D = 1.5–3.0)	0.55–0.6
Two or more layers of rock, (Hs/D = 1.5–6.0)	0.50–0.55

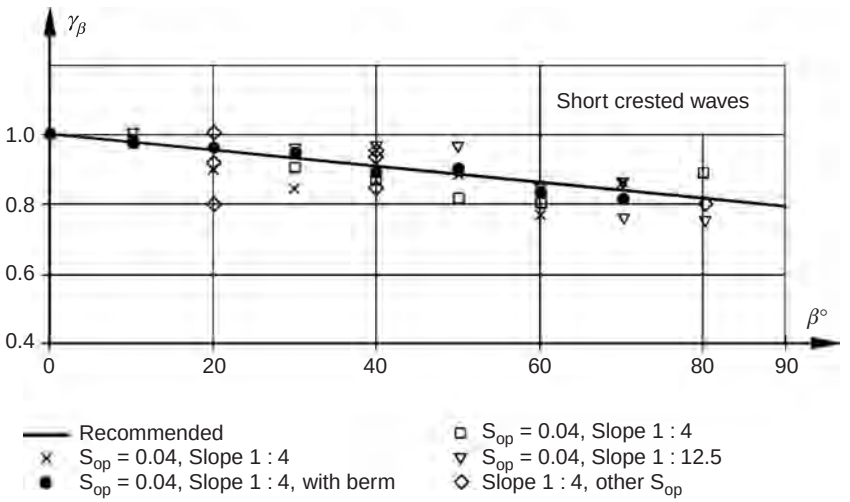


Figure 13.4 Influence of angle of incidence, β, and the directional spreading on run-up on smooth slopes (USBR, 2012, USACE, 2002, 2011).

Table 13.4 Values of variables A and C related to the surf similarity factor for peak wave heights, ξ_p (USBR, 2012a).

ξ_p *limits*	*A*	*C*
$\xi_p \leq 2.5$	1.6	0
$2.5 < \xi_p < 9$	−0.2	4.5

13.1.4.6 *Wind set-up*

The wind set-up (S) in metres is:

$$S = (V_W^2 F)/62000D \tag{13.8}$$

where
D = average water depth along the central radial (m) with an emphasis given to depths within a few km of the location for which set-up is being computed.

The wind set-up should be added to the wave run-up. It will be noted that, apart from long shallow reservoirs subject to high winds, the effect is very small.

13.2 SLOPE PROTECTION

13.2.1 Upstream slope protection

13.2.1.1 *General requirements*

The upstream slopes of earthfill on earth and rockfill dams need protection from erosion by wave action on the reservoir. Earlier dams were often protected by hand placed rock, but modern dams are generally protected by dumped rockfill, known as rip-rap.

Rip-rap comprises quarried blocks of rock which have to be:

- Large enough to dissipate the energy of the waves without being displaced.
- Strong enough to do this without abrading or without breaking down to smaller sizes.
- Durable enough to withstand the effects of long term exposure to the weather and varying periods of inundation without becoming weaker and, hence, wearing or breaking down to smaller sizes.

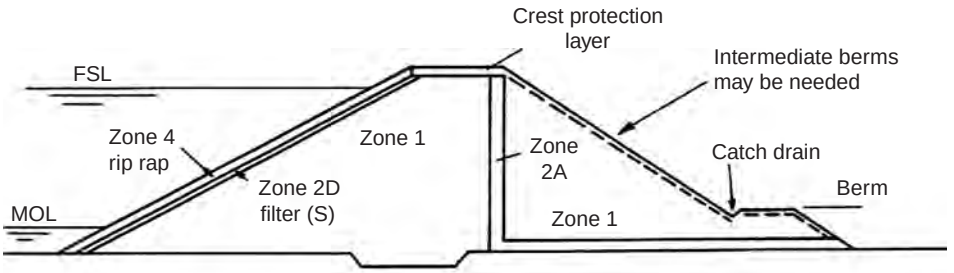
For earthfill dams, the rip-rap is constructed as a separate layer and should be underlain by a filter to prevent erosion of the earthfill through the rip-rap as shown in Figure 13.5(a). Alternatively a wider zone of quarry run rockfill with the larger rocks pushed to the upstream slope may be adopted. For earth and rockfill dams the rip-rap is often obtained by pushing the larger rock from Zone 3B to the edge, although in some very large rockfill dams large rocks have been selectively placed by an excavator with a 'claw' attachment (e.g. Thomson dam in Victoria where granite was used for the rip rap zone as distinct from sandstone/siltstone used for the rest of the rockfill). Only in dams where severe wave action is anticipated and Zone 3B rockfill is too small or not sufficiently durable would a separate layer be placed.

Where the reservoir is operated so that the water level is maintained at a high level, i.e. there is a minimum operating level well above the base of the dam, it may be possible to provide lesser or no rip-rap protection on the lower part of the dam. Because wave action affects about two times the wave height below the water level, rip-rap should be provided to minimum operating level less two times the design wave height. A small berm should be provided at this level to support the rip-rap layer and prevent undermining of the rip-rap by erosion when the reservoir first fills as shown in Figure 13.5(c).

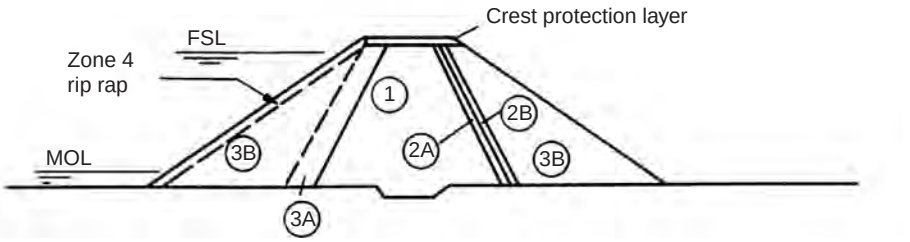
On some smaller dams, particularly those on mine sites where earth and rock moving equipment and waste rock are readily available, rip-rap may be designed in the knowledge that damage will occur due to larger waves or to breakdown of non durable rock, but that the damage can be readily repaired.

13.2.1.2 *Sizing and layer thickness*

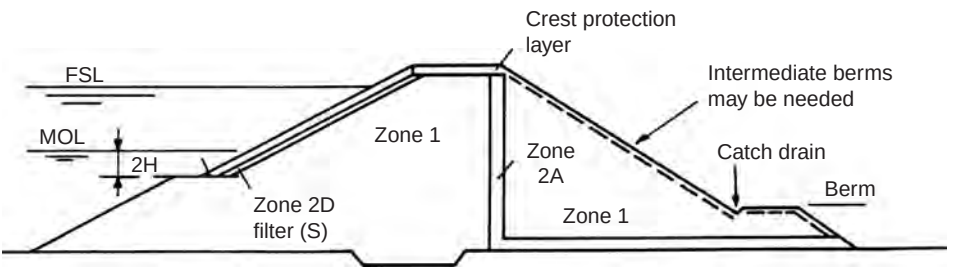
The sizing of rock needed for rip-rap, the layer thickness required, and filter layer requirements are determined from the size of waves expected on the reservoir and the nature of the earthfill or rockfill under the rip-rap. The procedure suggested for sizing



(a) Slope protection for earth fill dam



(b) Slope protection for earth and rock fill dam



Note: H = Design wave height

(c) Case with high minimum operating level

Note all sections diagrammatic – not to scale

Figure 13.5 Examples of upstream slope protection.

of rip-rap is that given in the US Corps of Engineers Shore Protection Manual (US Corps Engineers, 1984a; 2002). The steps involved are:

- Determine the design wave height H. This is taken on the average of the top 10% of the wave, which is 1.27 times the significant wave height determined as outlined in Section 13.1.4.3
- Calculate the weight of the graded rock in the rip-rap from:

$$W_{50} = \frac{\gamma_r H^3}{K_{RR}(S_r - 1)^3 \cot\theta} \tag{13.9}$$

Table 13.5 Rip-rap size versus wave height.

	Rip-rap size – (metres)			
	3H:1V slope		*2H:1V slope*	
Wave height (m)	D_{50}	D_{100}	D_{50}	D_{100}
0.5	0.19	0.27	0.21	0.30
1.0	0.37	0.55	0.42	0.63
1.5	0.55	0.82	0.63	0.95
2.0	0.73	1.10	0.84	1.26
2.5	0.92	1.38	1.05	1.58

where W_{50} = weight in kN of the 50 percent size in the rip-rap; γ_r = unit weight of the rip-rap rock substance in kN/m^3; H = design wave height in metres; S_r = specific gravity of the rip-rap rock relative to the water in the dam ($S_r = \gamma_r/\gamma_w$); θ = angle of upstream slope of the dam measured from the horizontal in degrees; K_{RR} = stability coefficient = 2.5 for angular quarried rock and non breaking waves.

The maximum weight of graded rip-rap (W_{100}) is $4W_{50}$ and the minimum 0.125 W_{50}. This is equivalent to the maximum size being 1.5 times the D_{50} size, and minimum size 0.5 times D_{50}. In practice if smaller minimum size rock is included it may be washed out under wave action. These values allow for less than 5% damage under the design wave. US Corps of Engineers (1984a) give factors which allow for greater damage.

For single size rip-rap, K_{RR} is replaced by K_D with $K_D = 2.4$ for smooth rounded rock and 4.0 for rough angular rock. These factors assume a rip-rap layer thickness allowing two layers of rock.

The equivalent sieve size of the rip-rap is approximately $1.15(W/\gamma_r)^{0.33}$. Where a thin rip-rap layer is being adopted, the average layer thickness can be calculated from:

$$r = nK_{\Delta}(W/\gamma_r)^{0.33} \tag{13.10}$$

where r = average layer thickness in metres; n = number of sub layers of rip-rap weight W in kN in the layer; $K_{\Delta} = 1.02$ for smooth rounded rock = 1.0 for angular quarried rock.

For rock with a specific gravity of 2.6 the estimated sizes of rockfill are as shown in Table 13.5. The wave height shown is the 10 percentile height. Values are given for upstream slopes of 3H:1V and 2H:1V.

Note that specifying rip-rap grading by linear dimension can sometimes lead to arguments about the acceptability of the rip-rap. This problem can be overcome by specifying the grading by mass of particles.

13.2.1.3 *Selection of design wind speed and acceptable damage*

On any project the decision has to be made on an acceptable degree of damage. This will influence selection of the design wind velocity and recurrence period. A useful guide

Table 13.6 Damage to rip-rap in percent as a function of design wave height (US Corps of Engineers, 1984a).

H/H_{DO}	*Damage (%)*
1.00	0–5
1.08	5–10
1.19	10–15
1.27	15–20
1.37	20–30
1.47	30–40
1.56	40–50

H = actual wave height; H_{DO} = design wave height.

can be obtained by considering alternative design wind velocity recurrence intervals, e.g. 1 in 10 year, 1 in 20 year, 1 in 50 year, 1 in 100 year, and assessing the effects of using a high recurrence design velocity using the damage estimates from Table 13.6. The figures in Table 13.5 are for quarried rock rip-rap.

The methods outlined in Thompson and Shuttler (1976), a CIRIA publication from the UK, are also valuable tools not only for designing the rip-rap but also for defining the expected damage.

What damage is regarded as acceptable will depend on the importance of the structure and the ease of access for repairs.

For existing dams, the authors' experience is that even rip-rap which is significantly smaller than a normal design has sometimes not suffered extensive damage after 30 years of operation. If the owner is willing to accept on-going maintenance, it is usually economic not to provide new rip-rap to modern design standards. Nevertheless it is important to recognize that inspection and maintenance are essential. If local failures are left without appropriate remedial works being done, the upstream edge of a crest and potentially part of the crest could be seriously threatened by erosion and slope instability of the over-steepened upstream slope, particularly in dams with continuously high storage level.

13.2.1.4 *Rock quality and quarrying*

The rock used for rip-rap must be able to withstand the repeated mechanical abrasion and wetting and drying action of the waves.

Rocks in the strong to extremely strong range are usually suitable for rip-rap if they can be quarried in intact blocks of sufficient size. Rock types which are commonly used when fresh include:

- Quartzite and sandstone.
- Limestone, dolomite and marble.
- Granite, diorite and gabbro.
- Basalt and andesite.
- Gneiss.

Most rocks containing siltstone, shale and claystone would be unsuitable for rip-rap because they would break down (slake) under repeated wetting and drying.

Ideally the rock should meet the durability requirements for concrete aggregates, but many rocks which do not meet these requirements have performed satisfactorily.

Laboratory tests which provide an indication of long term durability include:

- Apparent specific gravity and absorption.
- Petrographic examination.
- Methylene blue absorption.
- Accelerated weathering test such as wetting and drying, or sulphate soundness.

Fookes and Poole (1981) discuss the durability requirements for rip-rap.

Rip-rap is commonly obtained from rockfill quarries by stockpiling oversized rock from each blasting shot. If a special quarry is required for rip-rap, selection of potential sites must bear in mind that the spacing of persistent joints in the rock mass should be appreciably greater than the dimensions of the required blocks. It is possible to get a good initial indication of the likely sizes and durability of blocks obtainable from an outcropping source by observing the joint spacing in the outcrops and the size and condition of surface boulders and/or scree derived from them. Experience suggests that the size of excavated block will rarely exceed the joint spacing.

At the site, or sites, selected for detailed exploration a special objective of the exploratory programs should be determination of the pattern of joints in the rock mass, i.e. the number of sets of joints and the orientation, persistence and spacing of joints in each set. In this regard, geotechnical mapping of surface rock outcrops and trench exposures (even if extremely weathered) is important, as it is usually difficult to determine the joint pattern at depth from drill cores.

During core logging, particular attention should be given to:

- The location of any zones in which the rock appears to be affected by chemical alteration (e.g. in granitic and basaltic rocks – see Chapter 2, Sections 2.7 and 2.9.3 and Chapter 3, Section 3.2.3). Such zones often contain minerals which weather rapidly on exposure.
- The distinction between minor joints which may extend for less than 500 mm, and major joints which extend for many metres.
- The mineral type(s) which form the joint cements, and the apparent strength of the joints. In general, only quartz-cemented joints are likely to be strong enough to resist parting during quarrying and later when blocks are in service.

After logging, photography and sampling for strength and durability testing, it is useful to store the drill cores out in the open in a secure compound. Here they can be examined and photographed at regular intervals to record any deterioration. It is also possible to accelerate the 'weathering' processes by a program of wetting and drying.

13.2.1.5 *Design of filters under rip-rap*

See Section 9.2.5.2 for a discussion on the particle size distribution requirements.

Sherard *et al.* (1963) recommend that the thickness be selected after consideration of:

Table 13.7 Minimum thicknesses of filters under rip-rap (from Sherard *et al.*, 1963).

Wave Height (m)	*Minimum filter thickness (mm)*
0–1.2	150
1.2–2.4	225
2.4–3.0	300

- Size of wave.
- Gradation of the rip-rap. Rip-rap with less fines needs thicker filters.
- Plasticity and erodibility of the embankment earthfill. If the earthfill is well graded non erodible granular soil, it needs less protection than a fine silty sand or dispersive soil.
- The cost of the filter.

They recommend the minimum thicknesses shown in Table 13.7.

They indicate these should be absolute minimum thicknesses and that, if two layers of filters are needed, each should be at least 150 mm thick.

US Corps of Engineers (1984a) recommend that filter layers should be at least three 50 percent size stones thick, but not less than 230 mm.

The authors feel that the thicknesses in Table 13.7 seem reasonable for filters which satisfy no-erosion design rules, including limitations on maximum size particles. For filters which are quarry or pit run with particles up to 100 mm size, the minimum thickness would probably need to be at least 450 mm, so as to ensure that segregation of the larger particles does not leave areas with no finer particles to protect the soil under the filter.

13.2.1.6 *Use of soil cement and shotcrete for upstream slope protection*

In some locations rock suitable for rip-rap may not be available within economic haul distance and consideration may be given to the use of soil-cement as an upstream facing. Many dams in the USA have been constructed in this way.

ICOLD Bulletin 54 – Soil-Cement for Embankment Dams (ICOLD, 1986d) describes the use of soil-cement in some detail. The important features of the method are:

- The soil-cement is usually placed in horizontal layers as shown in Figure 13.6. The effective thickness normal to the slope has varied between 0.46 m and 0.76 m with 0.6 m being the most common. This gives a 2.4 m horizontal layer width for a 3H:1V upstream slope, allowing placement by trucks. Conveyor placement can be used to allow 0.46 m thickness, and on slopes 3H:1V or flatter, compaction up and down the slope can allow use of thinner covers. This may be acceptable for smaller dams.
- Portland cement is mixed with the available soil, conditioned to around optimum water content (standard compaction) and the soil-cement compacted in 150 mm to 300 mm thick layers to a density ratio of 98%. The soil cement is then cured under water spray.

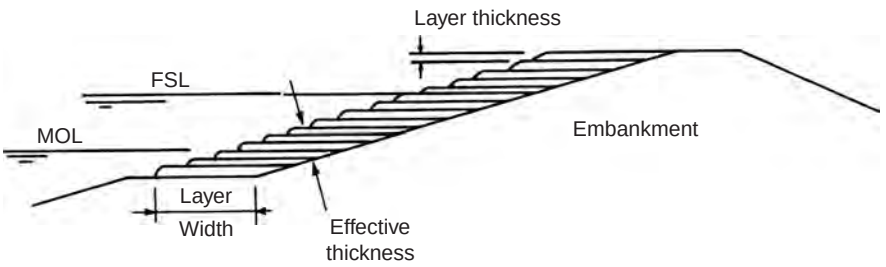


Figure 13.6 Use of soil-cement for upstream slope protection.

- A wide range of soils can be used but usually clayey sand and/or silty sand with 10% to 25% passing 75 mm. The Plasticity Index is usually less than 8%.
- The required cement content is determined by laboratory tests, which include wet-dry and freeze-thaw durability and unconfined compression strength. The requirements are detailed in ICOLD (1986d). Freeze-thaw testing would not seem appropriate in many countries with temperate or tropical climates. The USBR require an unconfined compressive strength of 4.1 MPa at 7 days and 6 MPa at 28 days. For field applications 2% cement (by weight) additional to that determined in the laboratory is used to allow for difficulties in mixing and compaction.

Soil-cement has proven to be a satisfactory slope protection over long periods - up to 30 years at least. Readers are directed to ICOLD (1986d) and more recent literature for additional details on the method.

13.2.2 Downstream slope protection

13.2.2.1 *General requirements*

The downstream slopes of earth and rockfill and concrete face rockfill dams are formed by rockfill. For these dams erosion of the face is not an issue and the requirement usually is simply to provide a uniform surface within the tolerance specified. Where no berms are provided access to survey markers or other instruments on the downstream face of rockfill can be difficult and consideration should be given to providing narrow berms at 30 metre maximum vertical intervals.

It is not unusual for contractors to go to some trouble nowadays to sort larger rocks on the downstream face using an excavator to give a pleasing appearance to the dam.

For earthfill dams, the downstream face is potentially erodible and considerable care needs to be taken to prevent erosion. This is done by:

- Covering the surface with a layer of rockfill or by establishing grass cover.
- Providing berms to limit the vertical distance over which runoff can concentrate.13.7
- Providing lined drains on the berms to catch the runoff and carry it to the abutments.

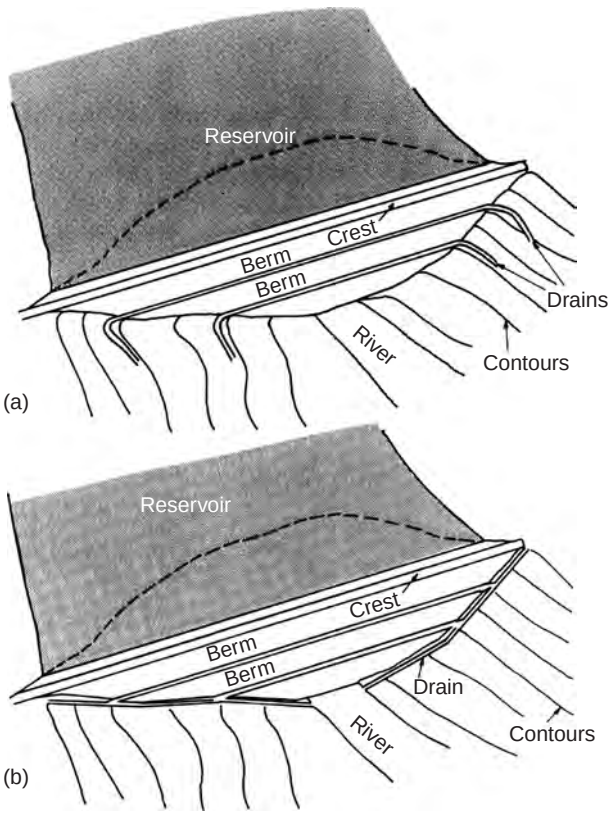


Figure 13.7 Berms and drainage for the downstream slope of earthfill dams where grassing is used to control erosion.

– Providing open lined drains at the contact between the abutment and the dam as shown in Figure 13.7(b), or the drains on the berms may be extended along the abutments as shown in Figure 13.7(a). In both cases erosion at the contact between the embankment and the abutment is being controlled.

The authors' experience is that berms should be provided at no greater than 10 m vertical intervals, although in arid climates berms may be unnecessary. A berm should be provided above the outlet of a horizontal drain as shown in Figures 13.5(a) and (c). This is necessary to prevent blockage of the outlet to the drain by soil eroded off the embankment.

When first constructed, drains will often block with eroded soil before the grass is well established and must be inspected and cleaned out regularly. Even when grass is established inspection and maintenance of drains are necessary to ensure they continue to function properly.

13.2.2.2 *Grass and rockfill cover*

The type of grass to be used is dependent on local conditions, particularly the climate and soil, and advice should be sought from local authorities such as the Soil Conservation Service. It is common procedure to provide a layer of topsoil and then to seed the slope using a bitumen hydro-mulch which provides initial protection against erosion before the grass establishes. Low native bushes also have been successfully used. It must be expected that the grass will need to be watered, at least until it is well established and that reseeding and repairs will be necessary. Grass cover locally reinforced with a patented erosion-resistant mat would be helpful at the dam/abutment intersection where run-off can concentrate.

Where there is an ample supply of rockfill and/or climatic conditions preclude the use of grass, dumped quarry run rockfill, placed directly on to the earthfill is usually a satisfactory way of controlling erosion on the downstream slope. USBR (1977) indicate that 0.3 m of rockfill usually is adequate although 0.6 m is usually easier to place.

13.3 EMBANKMENT CREST DETAILS

13.3.1 Camber

Embankment dams are subject to settlement after construction. In the case of earthfill this can be related to consolidation of the earthfill as pore pressures reach equilibrium but post construction settlement also occurs in rockfill, as with time high contact pressures between particles of rock are crushed.

Data on observed post construction settlement of earthfill and earth and rockfill dams are presented in Section 6.3.2, Table 6.8 and Figure 6.42.

From this it can be seen that most earth and earth and rockfill dams will experience less than 0.5% settlement in the first 10 years and a further 0.25% from 10 years to 100 years, i.e. a total of 0.75%.

Figure 13.8 shows the long term crest settlement rates for some concrete face rockfill dams, plotted as a function of dam height and the strength of the rock in the rockfill. The data is for well compacted rockfill.

It can be seen that, for high strength rockfill, an 80 m high dam could be expected to settle 0.3% in the 100 years after construction. This is consistent with Sherard and Cooke (1987) who suggest that settlements will generally be 0.15% to 0.3% in 100 years. They indicate that for dumped rockfill the crest settlements are 5 to 8 times that for compacted rockfill and the time dependent rate 10 to 20 times.

To maintain freeboard it is common to build the crest of the dam higher than the design level, i.e. to provide camber.

The camber relating to settlement of the embankment is best estimated by comparison with performance of other dams. Hence a fairly conservative approach would be to provide camber of about 0.5% to 1% for earthfill and earth and rockfill dams and about 0.2% to 0.4% for CFRD, depending on the height of the dam and strength of the rock in the rockfill. This is usually provided by over-steepening the upper slopes of the embankment, possibly by up to 0.25H:1V (e.g. for a rockfill dam with 1.5H:1V slopes, the upper slopes could be over steepened to 1.25H:1V). The camber changes as a

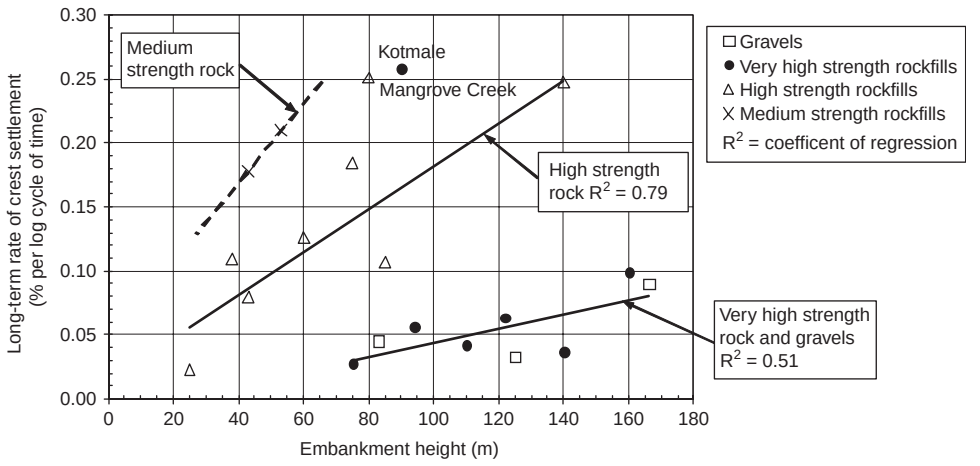


Figure 13.8 Long-term crest settlement rates versus embankment height for compacted rockfills (Hunter 2003, Hunter and Fell, 2003c).

smooth curve or series of straight lines along the dam crest and is provided proportional to the height of the dam above the foundation.

The camber relating to settlements of the dam foundation is best calculated using conventional soil and rock mechanics principles.

13.3.2 Crest width

The crest width has no appreciable influence on the overall stability of a dam and is determined by the minimum practicable width for construction purposes, and possible roadway requirements. Unless a wide road is needed for traffic purposes a crest width greater than 6 or 8 metres is seldom required, even for large dams. For small dams, a width of 4 metres will often be adequate. To achieve this, it is necessary to detail the zoning at the crest of the dam carefully.

Figure 13.9 shows an example of a crest detail for a central core earth and rockfill dam. The principles here are:

(a) The filters are taken as close to the crest as possible, to provide internal erosion and piping control under extreme flood conditions. Many older dams did not have this detail and, with revised flood estimates, it is now likely the crests will be tested by flooding.
(b) The filter width can be narrowed at the crest. This involves special placement of the filter. Filters should not be left out. Leaving out the Zone 2B filter removes protection against piping of the Zone 2A into the rockfill.
(c) The rockfill (Zone 3A) on the downstream side should also extend to the crest to protect Zone 2B from eroding.

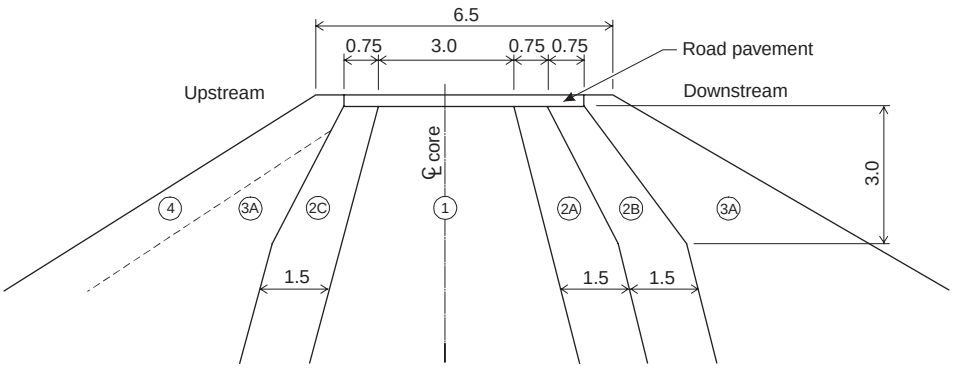


Figure 13.9 Example of a crest detail for a central core earth rockfill dam. All dimensions in metres. Steepening of the outer slopes for camber is not shown.

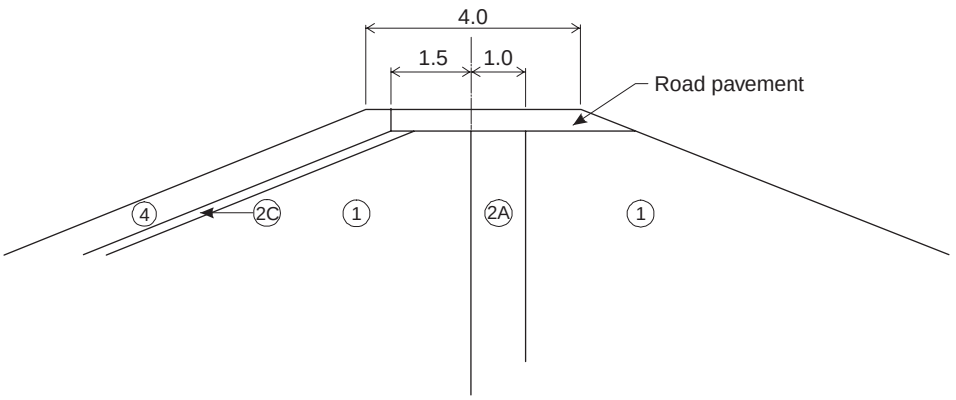


Figure 13.10 Example of a crest detail for an earthfill dam with a vertical filter drain.

Figure 13.10 shows an example of a crest detail for an earthfill dam with a vertical filter drain. In this case the width of the crest can be narrower than the earth and rockfill dam. It is assumed that the filter is constructed in the manner shown in Figure 9.28, i.e. by excavating through the previously placed earthfill. As for the earth and rockfill dam the filter must be taken as close to crest level as possible to provide protection from internal erosion and piping.

Any earthfill zone should be at least 3 m wide at the crest to allow compaction with normal rollers. No dam should have a crest width less than 3 m to allow vehicular access for maintenance purposes.

The crest is generally sloped towards the reservoir, to stop water ponding on it, and covered with a pavement to allow vehicles to traffic the crest. The authors' experience is that where no pavement has been provided, traffic causes wheel rutting and ponding of water on the surface which is undesirable. In this case it is wise to keep traffic off the crest by providing posts or a fence at each end. The pavement also serves the important function of reducing desiccation cracking of the core of the dam provided it

is constructed of non-plastic materials. The Zone 2A filter may be taken across the top of the core in a 200 mm thick layer to provide a more positive control of desiccation.

Section 10.9 gives details of embankment upgrades for internal erosion and piping, and for raising the crest. These include examples of the detailing at the crest of the upgraded dams.

13.3.3 Curvature of crest in plan

Some engineers have favoured curving the crest of earth and earth and rockfill dams in the upstream direction. As discussed by Wilson (1973), the concept is that, when the water load is imposed on a dam, the crest moves downstream as the water load is applied and, if the dam crest is curved upstream in plan, this can result in compressive strains along the axis of the dam. These compressive strains can counter tensile strains which are induced by settlement of the dam and hence reduce the likelihood of hydraulic fracture and leakage through the earthfill zones of the dam.

USBR (1977) suggest that for small dams the extra difficulty involved in constructing a curved axis is not warranted. Sherard (1973) indicates that the additional cost is very small, but the benefits of curvature are doubtful.

Sherard and Dunnigan (1985) consider that, with the greater confidence in the ability of well designed filters to control erosion, curvature of the dam axis is not necessary, even for high dams in steep valleys. The authors agree with this point of view.

13.4 EMBANKMENT DIMENSIONING AND TOLERANCES

13.4.1 Dimensioning

Embankment dimensioning is often done poorly by inexperienced engineers, because they fail to consider properly how the dam will be set out and constructed. Figure 13.11 shows some of the common errors. They are:

- Dimensioning height of dam rather than the reduced level of the crest including camber (RL b + camber). The height of dam varies across the valley and in any case is not known at any section prior to construction because the general foundation level is not known.
- Giving a reduced level at the base of the dam, RL t and/or depth of cutoff trench Z. These vary across the valley and are also not known before construction at any section. General foundation and cutoff foundation should be defined in geotechnical terms, not as levels and depths below ground surface or general excavation.
- Setting out the cutoff trench from the contact of earthfill zone with general excavation (i.e. point A) rather than as a fixed offset from the centreline. The position of A is not known before construction and is a variable distance from the centreline giving a 'meandering' location of the cutoff trench. The cutoff trench width is wrongly defined as shown, i.e. with width W at the cutoff foundation. In practice it is better to define the width D E which can be set after general excavation is completed, with an overriding dimension W_{min} to cover the situation where the depth of excavation to cutoff foundation is deeper than anticipated.

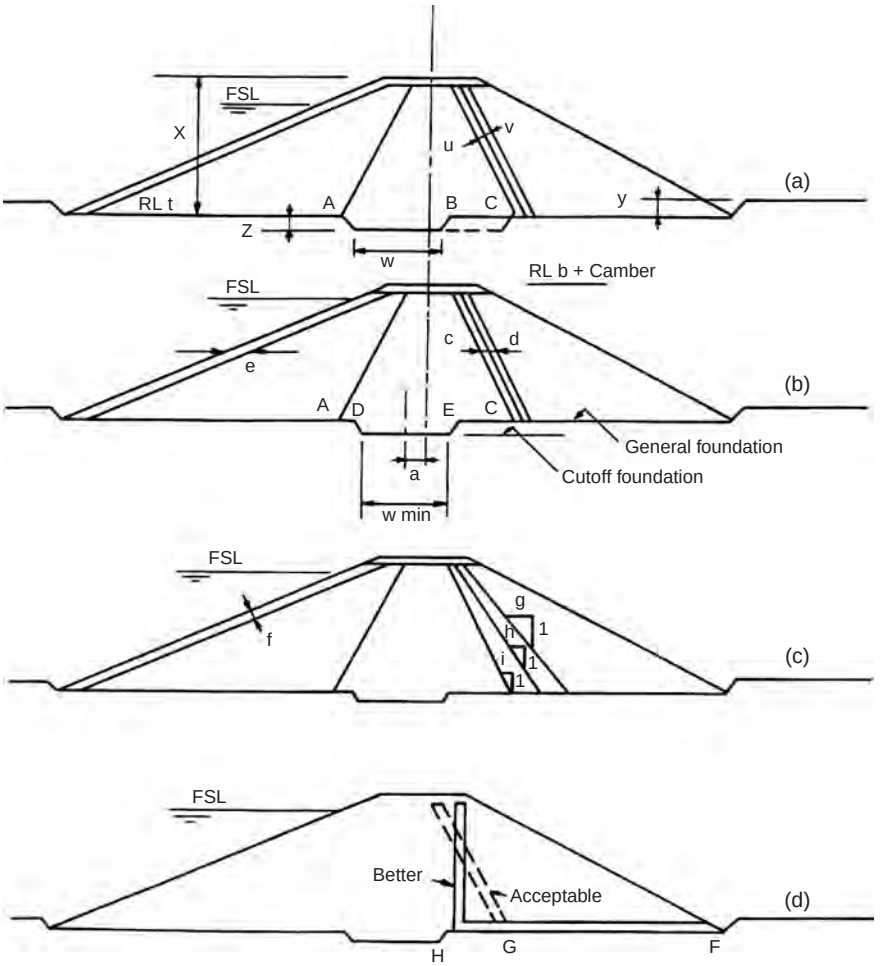


Figure 13.11 Issues in dimensioning embankment dams, (a) and (c) are usually poor practice; (b) and (d) are usually correct practice.

- Dimensioning filter and rip-rap layers as thicknesses normal to the slope (u, v and f) rather than as widths c, d and e. In most cases construction is in horizontal lifts so the widths are more appropriate.
- Dimensioning filter zones with varying widths using slopes g, h and i rather than as constant widths. There is no technical need for the filter width to increase with height; setting out is more complicated and volume of filters is increased by using the varying widths.
- Chimney drains should be vertical rather than inclined, as they are better constructed by digging back through compacted earthfill (see Figure 9.28) rather than placed concurrently with the fill.

Table 13.8 Tolerances for embankment construction.

	Towards axis of dam	*Away from axis of dam*
(A) Tolerances on zone boundaries		
(a) Outside faces of dam embankment		
Crest	Zero	250 mm
Downstream slope	Zero	500 mm
Upstream slope	Zero	500 mm
(b) Division lines between zones upstream of dam axis		
Zone 1 and Zone 2A	Zero	1000 mm
Zones 3A and 3B	1500 mm	1500 mm
(c) Division lines between zones downstream of dam axis		
Zone 1 and Zone 2A	Zero	1000 mm
Zones 3A and 3B	1500 mm	1500 mm
(B) Tolerances on widths and thicknesses		
(a) Width of filter zones and rip-rap		
Zones 2A, 2B, 2C	Plus	250 mm or 500 mm
	Minus	Zero
Zone 4	Plus	500 mm or 1000 mm
	Minus	Zero
(b) Thickness of horizontal filter drains		
Zones 2A, 2B	Plus	250 mm
	Minus	Zero

13.4.2 Tolerances

Specifications for construction of embankment dams usually include tolerances on the dimensions of the embankment. These reflect the need to ensure that, for example, the thickness and the widths of filter zones are maintained even if the position of the filter in the embankment varies slightly.

The tolerances shown in Table 13.8 would be reasonable for larger earth and rockfill dams with filters, such as those shown in Figure 1.2.

13.5 CONDUITS THROUGH EMBANKMENTS

The placement of conduits or outlet pipes through earth dam embankments is a common cause of piping failure and accidents, particularly for small dams which do not have filters. Foster *et al.* (1998, 2000a) record that historically about half of piping failures can be attributed to the presence of conduits.

Figure 13.12 shows a dam which has lost all the reservoir water in the large pipe which has formed around the outlet conduit.

The traditional approach to overcome this problem has been to provide concrete cutoff collars around the conduit. These lengthen the seepage path and penetrate into the fill, lessening the likelihood of a stress induced crack penetrating outside the collar. Figures 13.13 and 13.14 show typical cutoff collars for an outlet works conduit.

USBR (1987) indicate the cutoff collars were generally 0.6 m to 0.9 m high, 0.3 m to 0.45 m wide and spaced from 7 to 10 times their height along the portion of the

conduit within the core of the dam. For a conduit founded on soil, the collars should completely encircle the conduit. A bituminous or other joint filler is provided between the collar and the conduit to minimize stress concentration effects on the conduit.

While many dams have been built and operated successfully using this approach others have had problems.

Foster and Fell (1999) discuss the causes for this based on case studies, the literature and their experience. It is necessary to consider the mode of the piping (piping into the conduit, along and above the conduit or out of the conduit and the four phases of piping (initiation, continuation, progression and breach).

FEMA (2006) gives a comprehensive coverage of conduits through dam embankments and readers are encouraged to refer to that document.

13.5.1 Piping into the conduit

(a) *Initiation* – the conduit allows erosion into the conduit if it is cracked, corroded or joints have opened. This is most likely to occur if:

- Settlement of the conduit has occurred due to compressible (soil) foundations.
- The conduit is joined to a 'stiff' structure, e.g. outlet shaft or concrete core wall.
- Poor detailing of joints in design or construction.
- Water flows in the conduit under pressure fluctuate giving a surging effect.
- The conduit is old and corroded.

(b) *Continuation and progression* – the conduit facilitates continuation and progression by:

- Maintaining an open joint.
- Carrying away the soil which erodes into the conduit – so the seepage pressures do not have to erode the soil any great distance through the dam. However the progression of piping may be limited or slowed due to the limited width of the open joint or crack.

(c) *Breach* – the conduit does not in itself form a breach mechanism. Those failures which have involved erosion into the conduit have also involved erosion along the conduit. It is possible (probable?) that the initiation of erosion along the pipe was aided by erosion into the pipe. A breach mechanism can occur if the erosion progresses to form a large sinkhole which collapses at the crest leading to loss of freeboard.

13.5.2 Piping along and above the conduit

(a) *Initiation* – the conduit facilitates the initiation of piping by:

- Causing stress distributions due to the stiff conduit and its surround which lead to low principal stresses and hydraulic fracture. This is discussed by Sherard *et al.* (1972b) and Charles (1997). This can occur on the sides of culverts which are constructed in a trench. Melvill (1997) describes measurements on several South African dams. Knight (1990) measured very low



Figure 13.12 Piping failure of a dam around the outlet conduit.



Figure 13.13 Typical cutoff collar on outlet works conduit (USBR 1987).

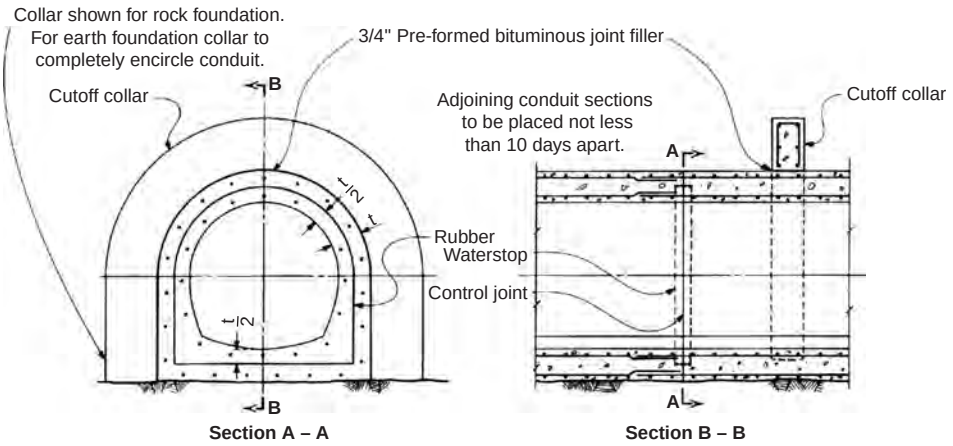


Figure 13.14 Typical control joint and cutoff collars details (USBR 1987).

horizontal stresses at Hinze dam in Queensland, Australia, sufficient to give virtually zero effective stress. Sherard *et al.* (1972b) point out that it can also occur where the concrete culvert, or concrete surround around a pipe, has a sharp corner. In this case piping can be expected above the culvert. Drying of the soil during construction can also cause cracks which allow initiation of piping. Sherard *et al.* (1972b) point out that, if the rate of filling is sufficiently slow, the soil swells, providing increased compressive stresses and preventing piping initiation.

– Making compaction of soil difficult, particularly if collars are provided at close intervals or the concrete is formed with corrugated steel sheet or other non-smooth formwork, preventing compaction of the soil adjacent to the conduit. Compaction will also be difficult if the trench for the conduit is narrow making compaction against the conduit difficult. See Figure 13.19). These can result in poorly compacted soil which is subject to collapse settlement on saturation forming a concentrated leak along the outside of the conduit.

Figure 13.15 shows some of these features. They are also discussed in Section 8.3.2.9 and Figures 8.18 and 8.19.

There is also some evidence that excavating a trench through an existing dam to install a conduit can lead to conditions conducive to initiation of piping.

The likely effects of this are:

– Possible drying shrinkage cracks in the sides of the trench.
– Different moduli of the compacted backfill compared to the original dam leading to cracking and differential settlement.
– Potential for poor compaction of the backfill in confined spaces.

Note that it is the continuity of the potential defect caused by a conduit and the excavation through the dam which is so critical to the initiation of piping.

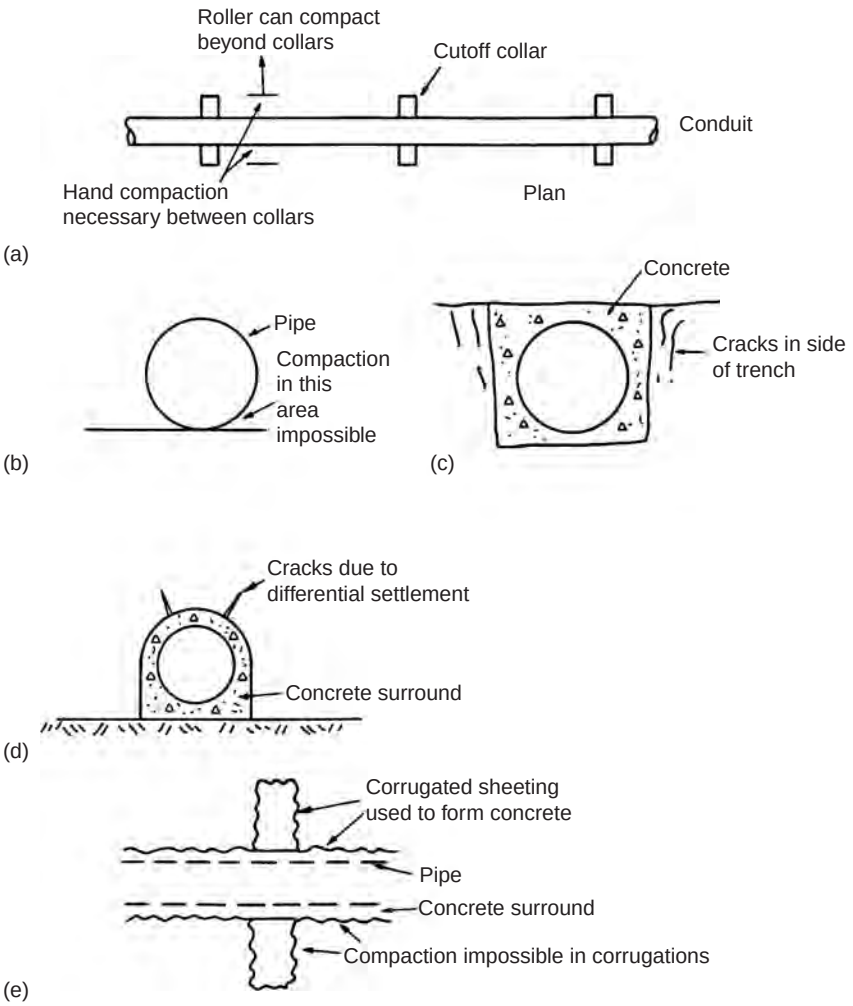


Figure 13.15 Some causes of piping failure around conduits: (a) Inadequate compaction due to the presence of cutoff collars; (b) Inadequate compaction under pipe; (c) Cracking in soil or extremely weathered rock in the sides of a trench; (d) Cracks due to differential settlement; (e) Use of corrugations or other roughening of the surface of cutoff collars or concrete surround.

(b) *Continuation and progression*. The conduit facilitates this by providing a 'side' or in the case of a circular outer shape, near its invert, a 'roof' to the potential erosion hole which will not collapse. Hence the likelihood of erosion developing beyond the initiation is greater than without a conduit.

(c) *Breach*. The conduit itself does not cause a breach mechanism which is mostly controlled by such factors as overall zoning. Flow from the conduit may facilitate removal of eroded or slumped material and therefore contribute in a secondary way.

13.5.3 Flow out of the conduit

(a) *Initiation*. The conditions conducive to flow out of the conduit (if it is cracked, or has open joints) are as for piping into the conduit.

In addition, the conduit must flow under pressure, either because it fills and pressurises under some conditions, e.g. large flood flows for a spillway outlet, or if the conduit is an outlet with the control valves on the downstream side.

(b) *Continuation and progression*. The flow out of the conduit leads to erosion of the soil along the conduit which may then lead to a piping failure mechanism or to increased pore pressures in the downstream part of the dam, leading to slope instability.

(c) *Breach*. The breach mechanism may either be as for erosion along the pipe or by slope instability, leading to loss of freeboard.

13.5.4 Conclusions

Sherard (1973) concluded that where conduits must be placed through earth dams:

- "*It is particularly important that the embankment adjacent to the conduit be placed at a relatively high water content and not be a soil susceptible to piping.*
- *Even in small, homogeneous dams where no chimney drain is installed it is advisable to provide a drain and filter around the conduit at its downstream end for the purpose of intercepting concentrated leaks which follow the conduit.*
- *In cases where the soil foundation is thick and compressible, it is not desirable to excavate a trench under the conduit and fill it with compacted earth.*"

Sherard and Dunnigan (1985) indicate that, because of concerns relating to compaction of soil adjacent to cutoff collars and the increased confidence in the use of filters to control erosion, they recommend that:

- No seepage cutoff collars be provided.
- Concrete surfaces surrounding conduits should be smooth and the sides should be sloped at 1H:8V or 1H:10V so that earthfill can be compacted directly against the concrete.
- A filter be provided to surround the downstream position of the conduit, i.e. underneath as well as on both sides and the top so that all potential leakage travelling along the concrete-earth core interface exits in a controlled manner.

At the ASDSO-FEMA 2000 workshop in Denver on issues, solutions and research needs related to seepage through dams, 29 participants, including senior dams engineers from the USBR, Corps of Engineers, FERC, major USA Consulting Companies and the first author, were unanimous in agreeing that cutoff collars should not be provided because they made compaction more difficult. FEMA (2006) also recommend that cutoff collars are not used.

Talbot and Ralston (1985) indicate that the Soil Conservation Service of the USA have replaced conduit cutoff collars with a filter diaphragm as shown in Figure 13.16.

They indicate that a single diaphragm is constructed around outlet conduits or other structures in the downstream section of the dam as shown in Figure 13.16. The

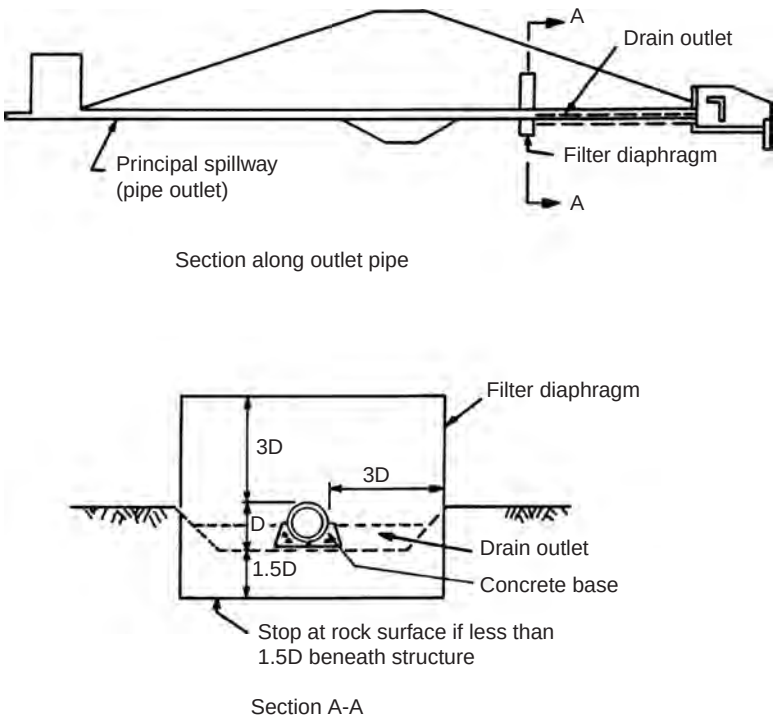


Figure 13.16 Filter diaphragm for seepage and piping control around outlet pipe (Talbot and Ralston, 1985).

diaphragm consists of a graded sand-gravel filter that projects outward a minimum of 3 times its diameter (for pipes) and extends 1.5 times its diameter below the pipe if the foundation is soil or erodible weathered rock. Where other embankment or foundation filter drainage systems are used, the diaphragm must tie into these systems to provide a continuous zone that intercepts all areas subject to cracking, poor compaction or other anomalies.

The 'drain outlet', is a layer, or layers of sand/gravel designed to act as a filter against the soil in the embankment and foundation, and with sufficient discharge capacity to transmit seepage intercepted by the filter diaphragm.

13.5.5 Recommendations

The authors recommend the following:

(a) Where practicable, avoid placing conduits through the dam, or on soil and erodible rock foundations, by using outlet tunnels in the abutments (for larger dams) or placing the conduit in a trench excavated into non erodible rock and backfilled with concrete as shown in Figure 13.17(a). Avoid deep, partly backfilled trenches, with steep sides as shown in Figure 13.17(b) because these will become

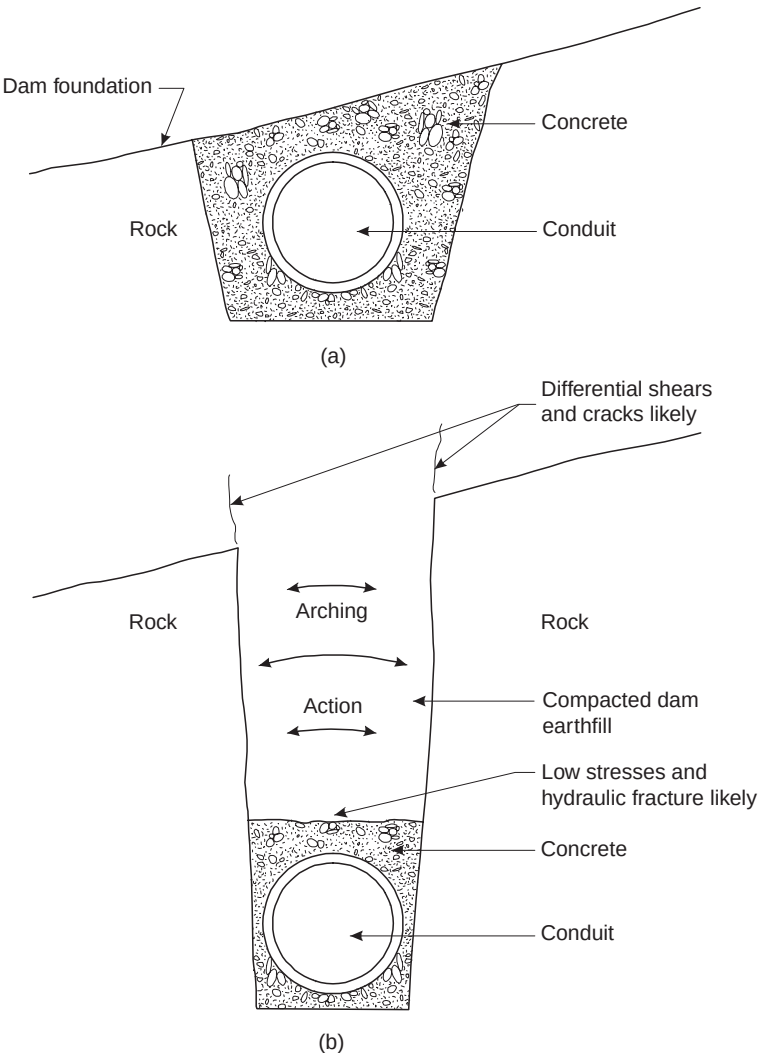


Figure 13.17 Conduit embedded in rock foundation (a) Good practice, (b) Poor practice.

zones of potentially poor compaction and low stresses conducive to hydraulic fracture and initiation of piping.

(b) Where conduits must past through the embankment, as will be the case for most smaller dams and many medium and large dams on soil foundations, the preferred detail is to provide filters for the dam core and to take these around the conduit. If the foundation is soil or erodible rock, the filter should be taken locally deeper into the foundation so as to surround the pipe in a similar way to the filter diaphragm in Figure 13.16.

Alternatively, for low consequence of failure dams, where filters are not being provided, or zoned earthfill construction is being used, and for flood control structures such as levee banks and dykes which have pipes or culverts through

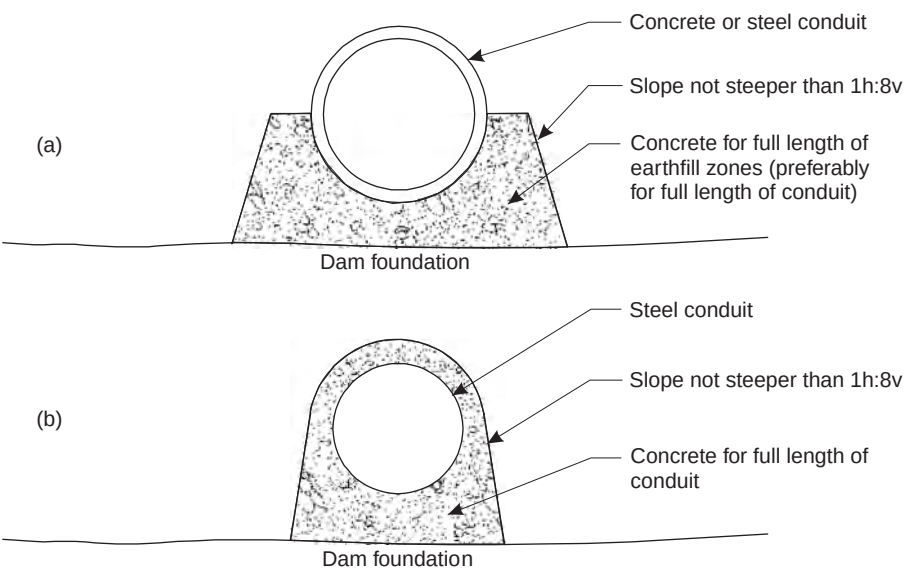

(a)
Concrete or steel conduit
Slope not steeper than 1h:8v
Concrete for full length of earthfill zones (preferably for full length of conduit)
Dam foundation
(b)
Steel conduit
Slope not steeper than 1h:8v
Concrete for full length of conduit
Dam foundation

Figure 13.18 Concrete support for (a) concrete or steel conduit strong enough to withstand earth pressure from the dam, (b) steel conduit requiring concrete surround to withstand earth pressure from the dam.

the levee, a filter diaphragm similar to that shown in Figure 13.16 should be provided.

(c) Concrete, steel, or metal pipes should be continuously supported on a concrete base similar to that shown in Figures 13.18(a) and (b). No cutoff collars should be provided. This is to allow compaction adjacent to the sides of the pipe, with rubber tyred rollers, rubber tyred construction equipment or hand held compaction equipment consistent with the strength of the pipe. The soil being compacted adjacent to the pipe should be wet of standard optimum water content to facilitate its being rolled into the irregular shape of the contact and to make it less likely to crack by differential settlement.

(d) Cast *in situ* conduits should be constructed as shown in Figure 13.19 with the sides sloping no steeper than 1 Horizontal to 8 Vertical (1H:8V), so compaction equipment can be used to the side of the pipe. The trench in which they are placed should be wide enough to allow for compaction and the batter slopes should be no steeper than 1H:1V so arching does not develop and lead to low stresses in the trench, conducive to hydraulic fracture and cracking.

(e) Care should be taken to prevent desiccation cracking in the base and sides of any trench into which a conduit is placed and any cracked soil should be removed immediately prior to backfilling.

(f) Where the only earthfill available is dispersive (Pinhole D1 or D2, Emerson Class 1 or 2), it would be wise, even with all of the above factors, to add 2% or 3% lime (by weight) to the backfill soil, carefully mixed, to render it non dispersive.

(g) Care should be taken to provide joint details in the conduit and between conduits and other structures, such as intake works, which will accommodate the

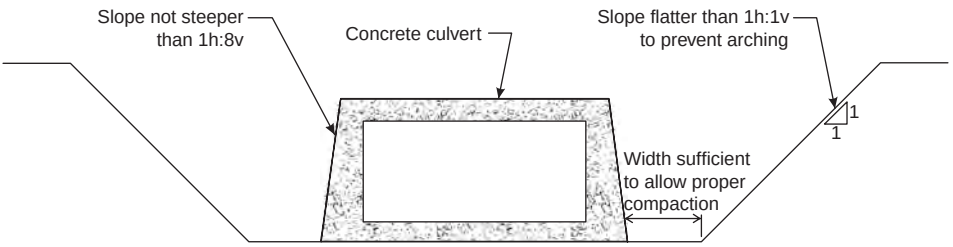


Figure 13.19 Trench and culvert details for cast-in-situ or pre-cast concrete culvert.

likely longitudinal and vertical deformations and prevent erosion of soil into the conduit or flow of water out of the conduit.

(h) Consideration should be given to the potential for corrosion of conduits and whether it is wise to rely on steel or corrugated steel pipes for long term operation. See Section 8.1.4 in FEMA (2006) for a discussion of the mechanics and consequences of corrosion of metal conduits passing through dams. The authors view is that corrugated metal pipes should not be used in dams given their poor performance history.

13.6 INTERFACE BETWEEN EARTHFILL AND CONCRETE STRUCTURES

13.6.1 Interface between retaining walls and embankment

Just as the contact between earthfill and outlet conduits can be a potential zone of weakness in an embankment, the contacts between earthfill and concrete structures, e.g. the wall of a concrete spillway, can be potential sources of cracking and piping failure.

Earlier practice was to construct a cutoff wall at right angles to the wall of the structure to penetrate into the earthfill and increase the seepage path. Figure 13.20 shows an example of this and shows the difficulties that are created because rollers cannot be used to compact the soil adjacent the walls.

For the example shown in Figure 13.20 the likelihood of a gap opening between the wall and the fill is increased because the foundation of the embankment slopes away from the wall. This is discussed further below.

In many existing well constructed dams where care was taken to compact around the cutoff wall by hand methods the dams have performed well. However the authors still regard the use of a cutoff wall as not good practise for new dams.

To overcome the difficulties in compaction Sherard and Dunnigan (1985) recommend that no cutoff wall be provided and that the contact be detailed as shown in Figure 13.21. This approach is recommended by the authors as it recognizes that the cutoff walls only hinder compaction and that cracking is inevitable leading to

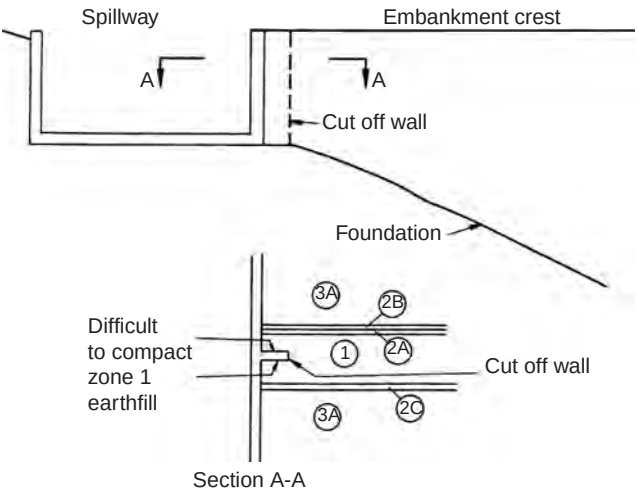


Figure 13.20 Contact between earthfill and spillway wall with a cutoff wall provided.

possible concentrated seepage. Such seepage should be controlled by well designed filters.

For this arrangement compaction of earthfill above standard optimum water content with rubber tyred equipment is possible right up to the wall. The wall is left smooth, off-form concrete, not sandblasted, chipped, painted or treated in any way. The downstream filters may be widened. Sherard and Dunnigan (1985) suggest that, if the Zone 2A filter contains a significant quantity of gravel sized particles, there is a danger the gravels could segregate at the concrete-filter interface leaving pockets of gravel without sand in the voids. In this situation they suggest placing an additional filter consisting only of sand upstream of the Zone 2A filter (as well as widening the Zone 2A and 2B filters).

If possible the wall is positioned as shown in Figure 13.21 so the foundation of the embankment does not slope steeply away from the wall.

Sherard and Dunnigan (1985) suggest that the angle α at which the dam axis intersects the wall (angle α in Figure 13.21) may be made say 80° to increase the pressure on the contact as the dam water load is applied. They point out, however, that many dams have been successfully constructed with $\alpha = 90°$.

It is recommended that, if highly dispersive soils are being used for construction, consideration should be given to adding lime in the vicinity of the contact with the concrete structure (in addition to the features detailed above).

Where the dam generally has no filters, it is strongly recommended that filters are provided in the vicinity of the contact.

Figure 13.22 shows two situations where settlement of the dam fill during or after construction, or in an earthquake, can cause a gap to form between the embankment fill and the spillway retaining wall. In Figure 13.22(a) the foundation slopes away at the base of the wall. In Figure 13.22(b) there is a change in slope of the back of the wall. Both situations are best avoided.

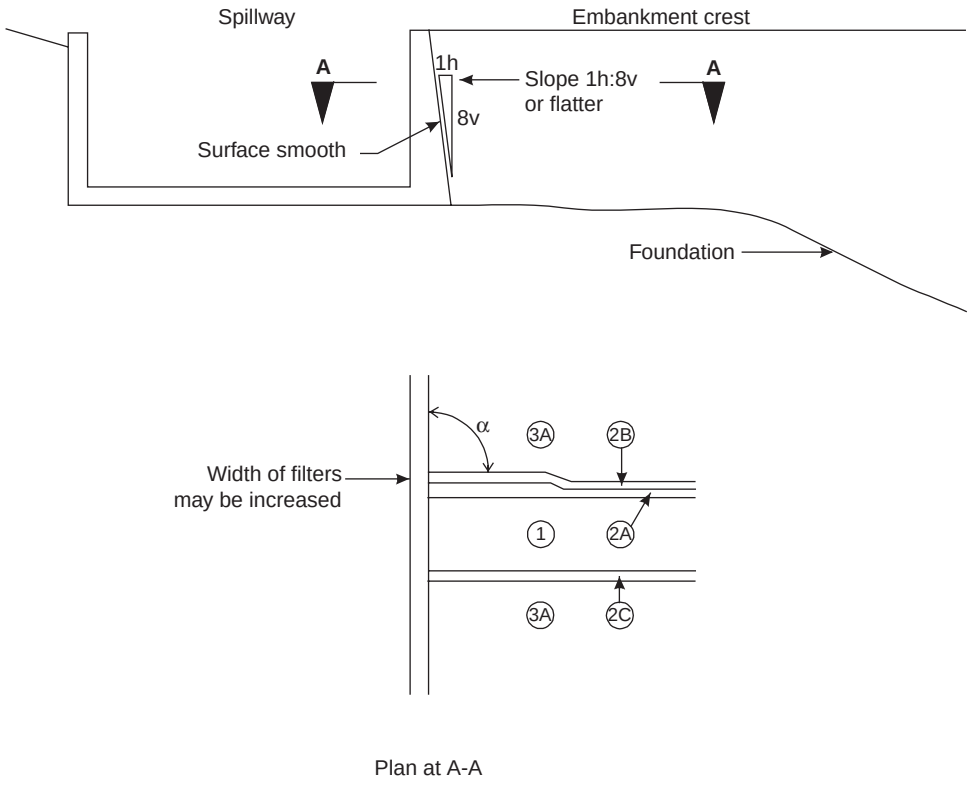


Figure 13.21 Good details for the interface between earthfill and concrete structures.

It should be noted that if a fine abutment contact soil is used against the wall (or a conduit in the previous section) the critical fine filter should be designed to suit this material as well as the general core material.

13.6.2 Interface between concrete gravity dam and embankment

Many dams have a concrete gravity section, often the spillway, with flanking embankments. For most the embankment is supported by retaining walls and the issues discussed in Section 13.6.1 apply.

For some dams the embankment is wrapped around the concrete gravity dam as shown in Figure 8.16. If this is done there is potential for a gap to open between the dam and the embankment fill resulting from settlement of the embankment during and post construction or under seismic loading.

The situation is made worse if the downstream slope of the dam is shaped with a change of slope as shown in Figure 8.15 or even worse with an overhang.

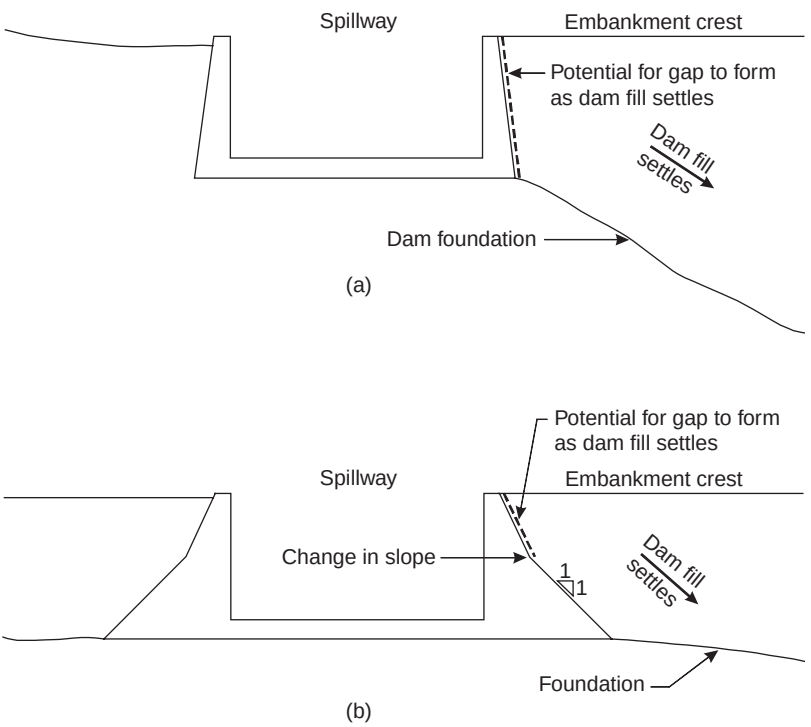


Figure 13.22 Situations where a gap may form between the dam fill and spillway wall: (a) Steep foundation adjacent spillway wall; (b) Change in slope of the retaining wall.

If this approach is being contemplated for a new dam it is suggested that the downstream slope be made uniform, and that similar care with filters be adopted as detailed in Section 13.6.1.

13.7 FLOOD CONTROL STRUCTURES

Flood control structures, e.g. flood storage basins and levee banks and dykes, have particular features which require consideration. These include:

- The embankments stand without water adjacent to them for long periods.
- Water levels rise quickly and usually fall within days or even hours.
- The consequences of failure may not be as critical as for other dams.
- The embankments may serve other purposes, e.g. as a parking area.
- The structures often have culverts or pipes passing through them, some of which may have been constructed after the embankment was completed, possibly without the knowledge of the relevant embankment owner.
- The foundations are usually soil and often these have sandy layers in them, so 'backward erosion piping, contact erosion, suffusion and concentrated leak erosion through fissures in the soil are an issue.

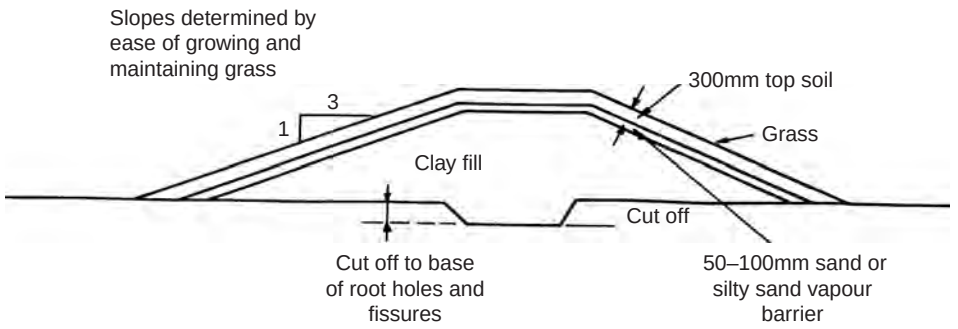


Figure 13.23 An example of the design for a levee bank or dyke in a low consequence of failure situation.

From a design viewpoint this has the following implications:

(a) The embankments may crack due to desiccation if built of clay soils.
(b) If the rate of rise of the water is rapid it is likely erosion will begin before cracks close by swelling of the soils adjacent to the cracks. If the rate of rise is slow cracks may close as the water rises.
(c) There are potential points of weakness around the conduits as described in Section 13.5.

The design of such structures, particularly levee banks and dykes, which are long, low structures, built with limited funds, is often a compromise between cost and the likelihood of failure. This has to balanced against the consequences of failure. Many levees and retarding basins with low consequences are built only to overtop a 1 in 100 or 1 in 200 AEP flood, so it is probably inappropriate to provide fully intercepting filters at great cost. Even so, filters should be provided around any conduits or other penetrations through the structure.

Figure 13.23 shows a possible detail for a levee bank where the consequences of failure are low and there is no 'heave' or 'blow-up' risk. The section is homogeneous earthfill. Cracking is controlled by providing a 50–100 mm thick sand layer which acts as a vapour barrier. A road surface on the crest would also assist in this. The cutoff is taken below the base of cracked and fissured soil.

If the soil is dispersive, it should be modified with lime or gypsum to render it non-dispersive.

Compaction control should be as rigorous as for any dam and all conduits, culverts, pipes and walls should be treated with a filter diaphragm as discussed in Section 13.5. This should give a likelihood of piping failure less than that of overtopping.

The embankments are unlikely to reach the steady state seepage condition (or instantaneous drawdown), so slopes can be steeper than a normal dam. Hence from slope stability considerations, embankments built of clay soils can stand at 1.5H to 1V. Silty sand, or sandy soil embankments will, however, saturate quickly and require flatter slopes.

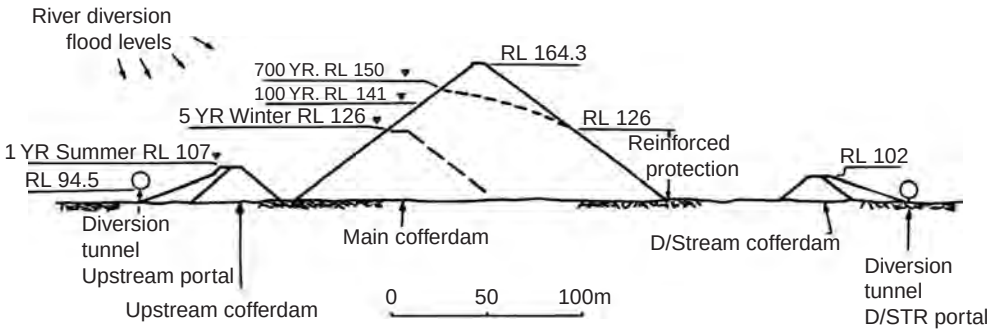


Figure 13.24 Coffer dams for river diversion.

From the erosion control and maintenance viewpoints, slopes of 3H to 1V or flatter are preferable and this may override stability considerations.

The partial saturation effect may be more important for levee banks on the side of a river if it can be shown that the levee will not saturate. However, care must be taken as most levee bank failures occur on drawdown of the flood.

It should be recognised that use of such a cross section does have risks of failure higher than for well designed dams. Where practical and warranted from consequences of failure viewpoint, it would be better to use a zoned earthfill cross section, or earthfill with vertical and horizontal drains.

For flood control structures, such as retarding basins, in urban areas, the consequences of failure should be assessed and the embankment section selected accordingly. Where there is a potential for loss of life, the embankment should have proper internal erosion and seepage control including fully intercepting filters and adequate spill capacity. Many existing retarding basins now may fall into high or even extreme consequence categories; some of these are now being upgraded by installing filter protection and increased flood capacity.

13.8 DESIGN OF DAMS FOR OVERTOPPING DURING CONSTRUCTION

13.8.1 General design concepts

When embankment dams are constructed on a river it is necessary to divert the river from the river bed during construction (of the dam or a culvert along the lower part of one of the abutments). This is usually done by excavating a diversion tunnel or a channel through an abutment of the dam and constructing a coffer dam upstream of the embankment to divert the river into the diversion tunnel or channel. This upstream coffer dam may be supplemented by a higher coffer dam which is incorporated into the main embankment as shown in Figure 13.24.

When designing coffer dams a balance is reached between the height (and therefore cost) of the coffer dam and the recurrence period of the flood which will overtop the dam. Also important are the consequences of overtopping in potential damage to the

construction works, delay to construction schedules and potential for damage or loss of life downstream. Of particular concern is the situation if the coffer dam fails, giving a substantial flood wave.

In the example shown in Figure 13.24, the upstream coffer dam has been designed to divert the 1 year summer (dry season) flood and the main coffer dam the 5 year winter (wet season) flood. There is therefore some likelihood that it will be overtopped.

It has been common practice in Australia since the late 1960s to provide protection to the coffer dam and/or to the main embankment of rockfill dams to prevent failure in the event of overtopping. This is done by covering the downstream face of the coffer dam or the main embankment with steel mesh or rock filled gabions, which are anchored into the slope with steel reinforcing rods. If this reinforcement is provided on the main embankment it can be designed so that much greater return period floods can be accommodated, e.g. 1 in 100 year or 1 in 700 year or even lower frequency floods. Whether these are acceptable should be determined in a risk analysis framework.

The use of such steel mesh reinforcement has allowed use of significantly smaller diversion tunnels and upstream coffer dams, with resultant savings in project cost. The design, construction and operation of steel mesh reinforcement are described in ANCOLD (1982) and ICOLD (1984). As detailed there, up to 1982 the method had been used on 50 dams; 41 in Australia, 9 in other countries.

The following outlines only the broad principles and for design purposes reference to the ANCOLD or ICOLD publications and more recent publications relating to overtopping design, and for flow through rockfill such as NVE (2012) and as detailed in Section 10.9.2 is recommended.

13.8.2 Types of steel mesh reinforcement

The technique evolved over several years, being modified to take account of lessons learnt from failures and to reduce the costs involved. Some early systems failed during operation, due to:

- Damage of relatively lightweight steel meshing, by floating logs, or by boulders from the surface of the dam, e.g. Cethana Dam (HEC, 1969)
- Erosion of the contact between the steel meshing and the downstream abutment due to inadequate anchorage, e.g. Xonxa Dam (Pells, 1978).

A design which took account of these early problems, and successfully withstood 33 hours of overtopping, up to 2.5 m deep when the dam was 20 m high, was that employed at Googong Dam (Fokkema *et al.*, 1977).

Figure 13.25 shows the meshing system for Googong Dam.

There are several features of the design:

- Continuous concrete protection around the toe where the mesh joins the foundation. This is a critical potential weak link because of concentration of flows down the groin of the dam.
- The cranked anchor system for the avoidance of progressive failure.
- The combination of a medium duty surface mesh with small opening size and an overlay of fairly closely spaced (500 mm centres) sloping bars to protect the mesh.

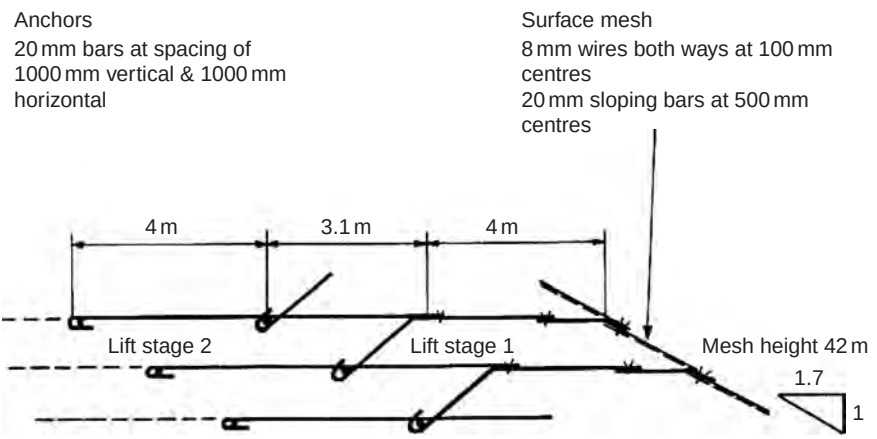


Figure 13.25 Steel mesh reinforcement for Googong Dam (Fokkema *et al.*, 1977 and ANCOLD, 1982).

- The return of the sloping bar into the fill for tensioning to the anchor prior to welding the connection between the two.
- Infilling voids in the face rockfill with concrete to lessen the chances of individual rocks being plucked out (but not too much so as to lower the through flow capacity of the rockfill).
- Being able to secure the protection at the top of each layer as it is approaching completion, with the use of temporary gabions on the abutment section of each layer to direct the flow to the centre of the dam.

The Hydro-Electric Commission of Tasmania carried out a significant amount of developmental work in steel mesh reinforcement. Figures 13.26, 13.27 and 13.28 show systems they developed to cope with their particular problems – regular overtopping, particularly of coffer dams, and heavily timbered catchments, which give potential for damage during overtopping by floating logs.

These systems all use cylindrical gabions fabricated from 50 mm square mesh with 5 mm wires both ways (4 mm for Mackintosh Coffer Dam).

For the Mackintosh Coffer Dam (Figure 13.26), a concrete slab was placed over the rockfill crest to secure the top row of gabions and prevent overflowing water from entering the downstream rockfill zone through the crest. Model tests were carried out to measure the pore pressures used in stability analysis which showed that anchor bars were not required. The coffer dam was overtopped by small floods several times without any damage being sustained. A similar system has been used on other HEC dams.

For Mackintosh Dam the downstream face of the dam was protected to a height of 21 m. As shown in Figure 13.27 this took the form of a vertical wall of gabions anchored to dowels grouted into the rockfill. The protection was made vertical to keep the many floating logs that were expected to be carried during floods clear of the face. The triangular wedge of rockfill forming the toe of the dam was placed after

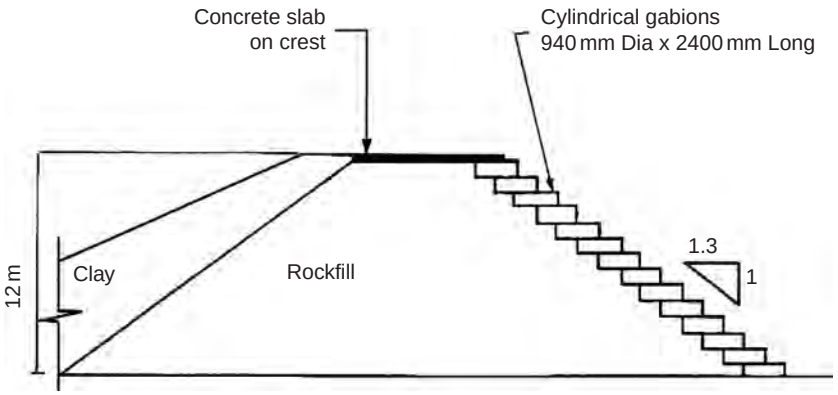
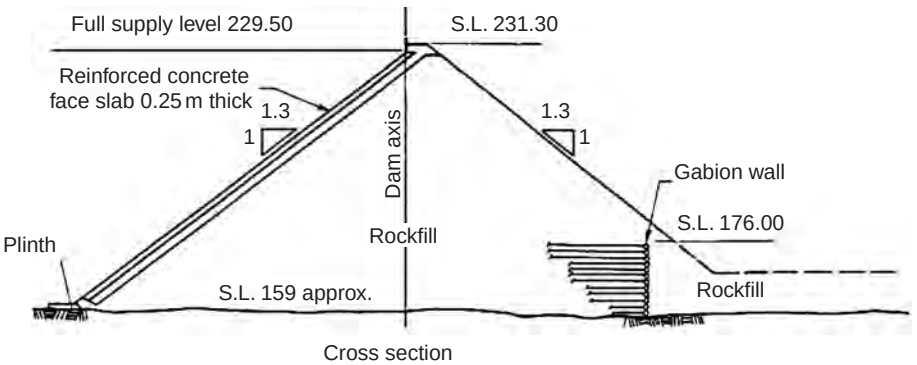


Figure 13.26 Steel mesh reinforcement for Mackintosh Coffer Dam (ANCOLD, 1982).



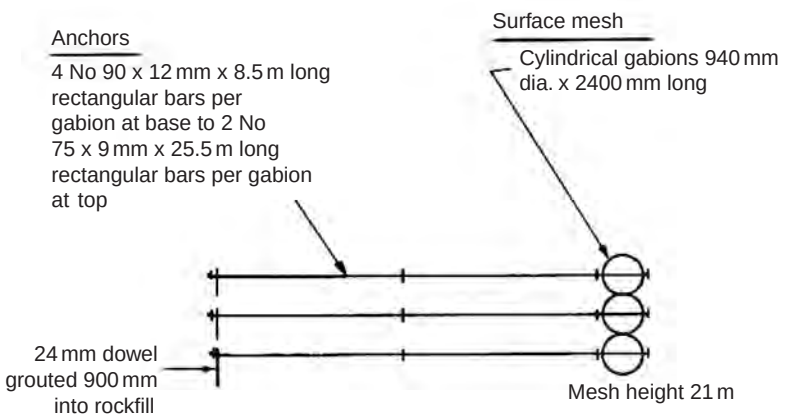

Anchors
4 No 90 x 12 mm x 8.5 m long
rectangular bars per
gabion at base to 2 No
75 x 9 mm x 25.5 m long
rectangular bars per gabion
at top
Surface mesh
Cylindrical gabions 940 mm
dia. x 2400 mm long
24 mm dowel
grouted 900 mm
into rockfill
Mesh height 21 m

Figure 13.27 Steel mesh reinforcement for Mackintosh Dam (ANCOLD, 1982).

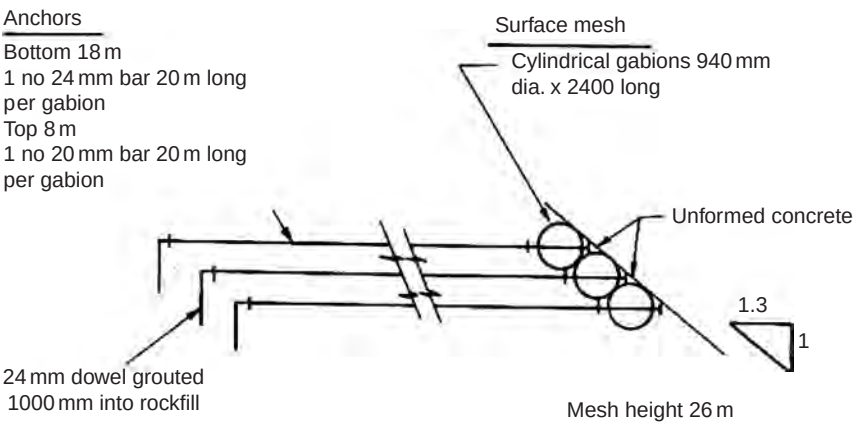


Figure 13.28 Steel mesh reinforcement for Murchison Dam (ANCOLD, 1982).

the embankment was above the 100 year flood level. The dam was overtopped once during construction but only to a depth of 300 mm.

The system was found to be costly mainly because of the very substantial anchors required and the full strength bolted connections. Although this type of connection was used in preference to a welded connection to avoid delays in wet weather, the large number of anchors and connections slowed down the installation of the system and delayed embankment construction.

For Murchison Dam, the gabions were placed on the same slope as the face of the dam as shown in Figure 13.28. The cylindrical gabions could be installed rapidly and did not control the rate of embankment construction. The exposed upper surface of the gabions on the dam face was protected with unformed concrete. There was no overtopping event during construction, although flow through the rockfill occurred in a flood which ponded above the upstream coffer dam when the dam was well above the mesh level. A similar system has been used on later HEC dams.

Another application of the Googong type of meshing system is shown in Figures 13.29, 13.30 and 13.31 for Clarrie Hall Dam. This incorporates a rather simpler system of anchorage than the cranked bars used at Googong and pays particular attention to preventing erosion at the abutment of the dam. Figure 13.29(b) shows the detail in the river and Figure 13.29(c) the abutments.

A critical issue in steel mesh reinforcement for each layer is to keep the crest level as uniform as possible, and to be able to tie down the upper surface. Figure 13.31(b) shows gabions which were used in the event of imminent overtopping in lower parts of the embankment crest.

13.8.3 Design of steel reinforcement

The design principles are:

(a) Control ravelling of the rockfill by steel mesh or gabions.

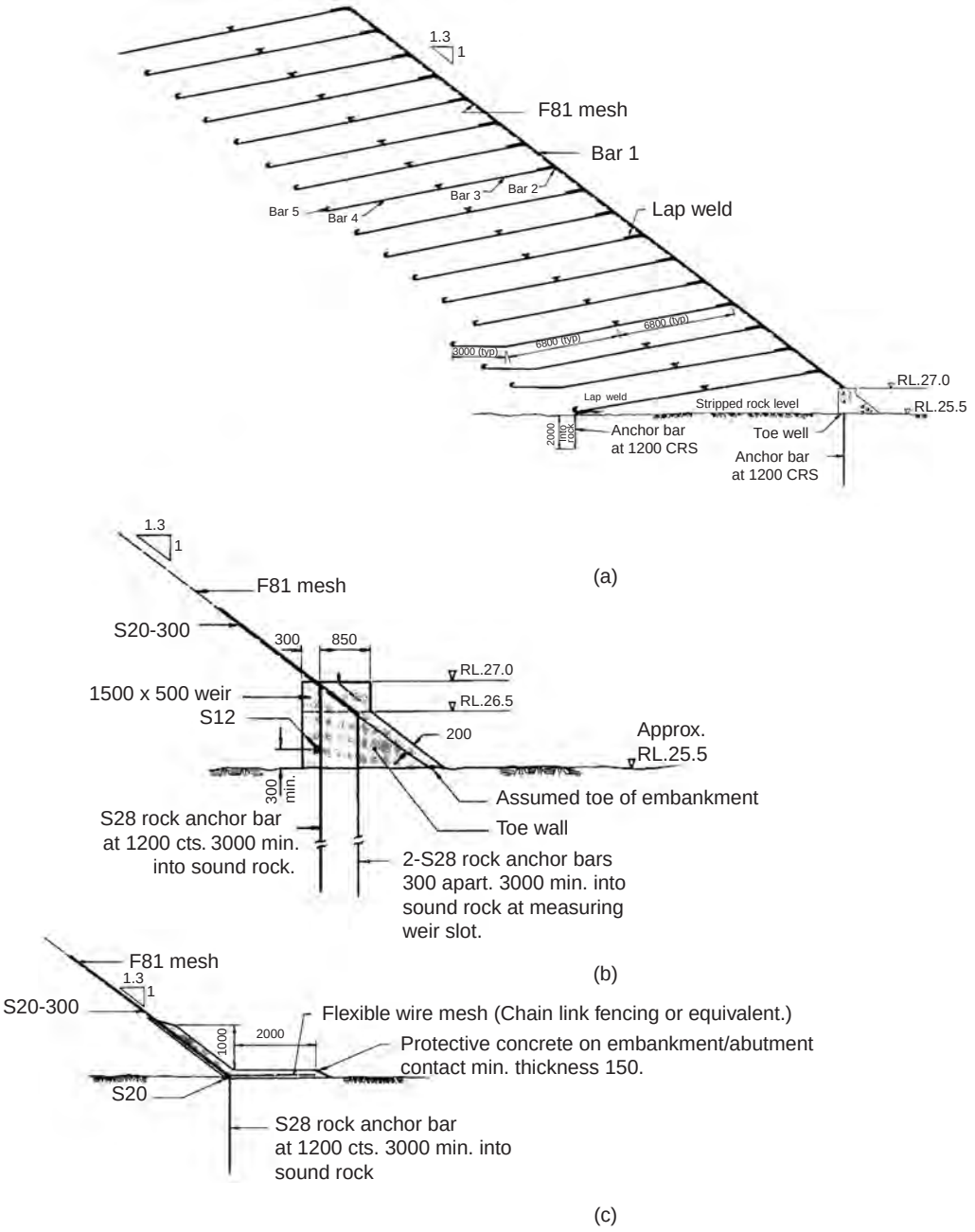
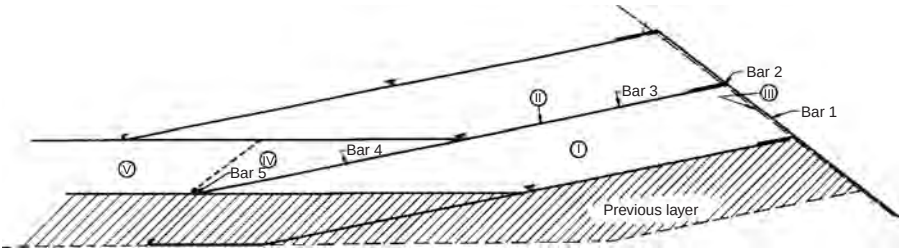


Figure 13.29 Steel mesh protection for Clarrie Hall Dam, (a) meshing layout; (b) abutment detail, river section; (c) abutment detail, other than river sections (PWD of NSW, 1981).



Construction sequence diagram

Notes:
Construction sequence

1. Before commencement of any meshing prepare and have on hand 24 gabions 2.0 m long × 1 m wide × 0.5 m deep.
2. Construct rockfill windrow (I) compact and trim face.
3. Construction on windrow shall not proceed more than one shift in advance of mesh placement.
4. Place and Tie F81 face mesh (III) place Tie bars (II) (bars 3 and bar 4 and anchor bar 5).
5. Place rockfill (IV) over bar 4 and proceed with placement and compaction of layer (V) place face cover bar (Bar 1) and horizontal bar (Bar 2) and weld cover bar (Bar 1) to previous layers.
6. Weld bar 1 to bar 2.
7. Bend bar 1 to align with bar 3. Tension bar 3 to 5 kN against bar 1 and lap weld.
8. In event of a threatened overtopping. Do not advance windrow further. Complete trimming and meshing including tensioning and welding over exposed windrow. Place protective gabions along windrow edge and weld to mesh. Tie gabions together.

Figure 13.30 Steel mesh reinforcement for Clarrie Hall Dam – construction sequence (PWD of NSW, 1981).

(b) Protect the mesh from damage from floating debris by placing reinforcing bars up and down slope over the mesh; or construct the slope very steep with gabions so debris shoots out beyond the mesh/gabions; or cover (in part) with concrete.

(c) Prevent damage to the mesh from rockfill washing off the top of the dam by ensuring that the meshed section is always the highest point on the dam section. Rapid 'closure' methods are needed if flooding is predicted.

(d) Anchoring the mesh into the dam in such a way that the anchors are always protected from erosion by the next layer of fill i.e. crank-shaped anchors (Googong); inclined anchors (Clarrie Hall); anchors fixed to grouted dowels (Mackintosh).

(e) Anchor length is determined by slip circle or wedge analysis, with anchor forces included. Pore pressures are estimated from design charts (see Fitzpatrick, 1977 and Lawson, 1987), seepage flow nets or preferably by hydraulic model tests. These must allow for the likely anisotropy of the permeability of the rockfill due to finer material at the top of each layer created by placing and compaction.

(f) The mesh is anchored into the abutment to prevent it being eroded out. Flow concentrates in this area during overtopping. Concrete protection may be provided (as in Clarrie Hall).

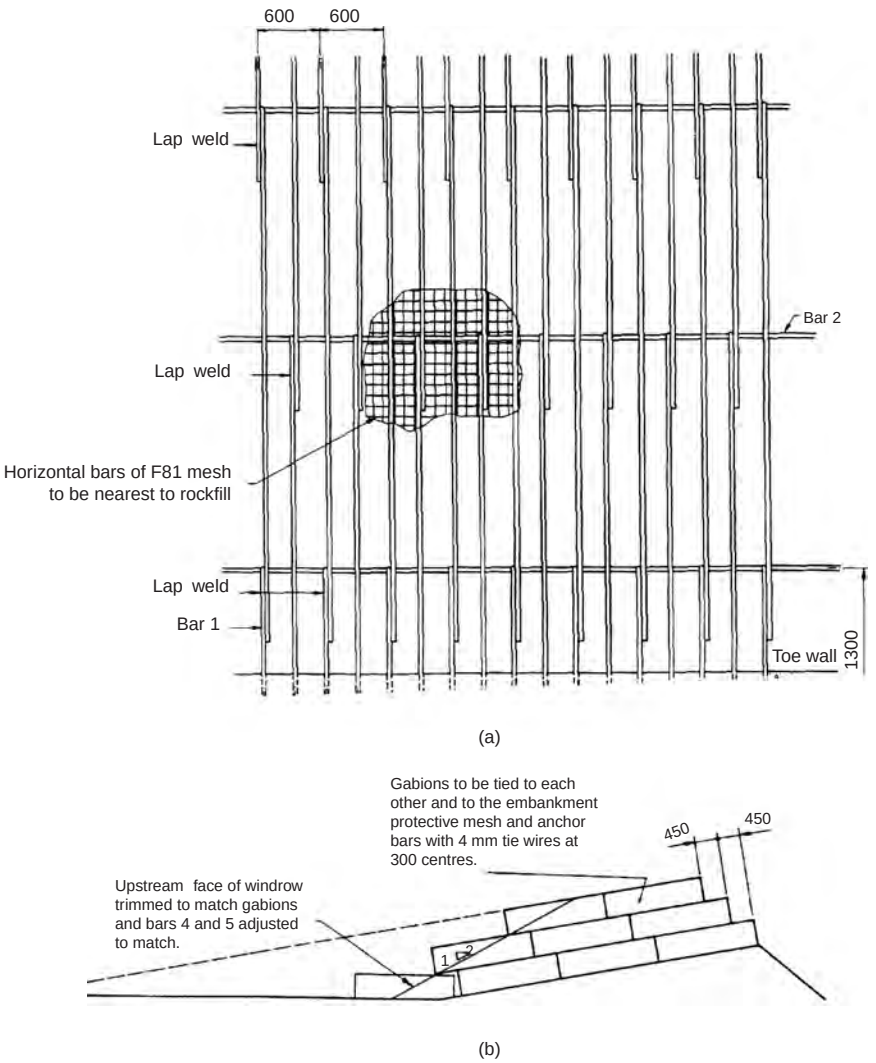


Figure 13.31 Steel mesh reinforcement for Clarrie Hall Dam, (a) dam detail; (b) crest gabions.

Detailing is vitally important and reference should be made to the papers referenced in ANCOLD (1982) and ICOLD (1984).

13.9 DESIGN OF RIP RAP FOR MINOR OVERTOPPING OF LEVEES OR SMALL DAMS DURING FLOODS

Levees and other low embankments may withstand shallow overtopping for sustained periods if the downstream slope is protected by riprap.

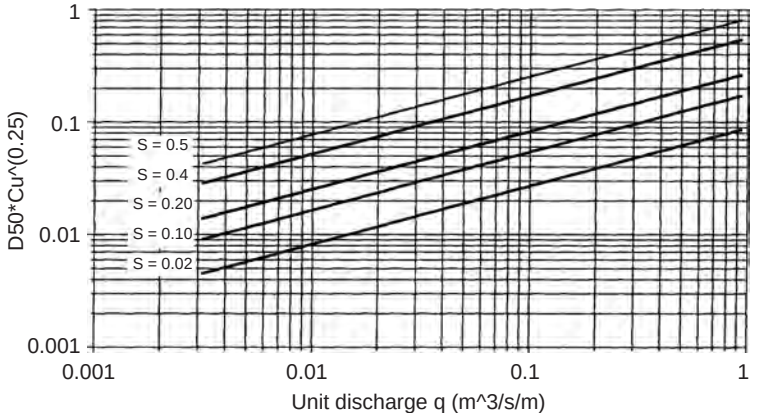


Figure 13.32 Design curves to size riprap protection on embankments of various slopes. The curves represent the point of incipient failure. No safety factor has been included. (Frizell *et al.*, 1998).

Methods for design of the riprap are summarized in Abt *et al.* (2013). They gathered test data from the several laboratory studies which have been carried out and used these to check the performance of 21 available design methods.

They concluded that the methods of Khan and Ahmad (2011) and Chang (1998) gave the best fit overall to the data for angular rock. The Khan and Ahmad (2011) relationship is:

$$D_{50} = 0.66t^{0.58}S^{0.22}C_u^{-0.45}q_f^{0.22} \tag{13.11}$$

where
D_{50} = size of rock in the riprap for which 50% is finer (mm)
t = thickness of riprap layer (mm)
S = embankment slope relative to the horizontal (ratio of vertical to horizontal)
C_u = Uniformity coefficient of riprap (D_{60}/D_{10})
q_f = overtopping discharge/unit length of levee or dam (m^3/sec/m).

This equation has no factor of safety. It was based on a multivariate regression analysis of published data. Care should be taken in applying this method because the data used is from several sources and the limitations are not clearly defined.

Frizell *et al.* (1998) describe the results of large scale laboratory experiments carried out by USBR on rip rap subject to overtopping flow. They tested riprap with D_{50} sizes up to 655 mm. Their method was developed for angular riprap supported upon a bedding layer and applies only for flows which are carried in the interstitial flow of the rip rap. No factor of safety applies.

The method involves:

- Make a first estimate of the required D_{50} size (m) of the riprap for the design overtopping discharge/unit length of levee or dam, q (m^3/m) and an assumed uniformity coefficient C_u and embankment slope S using Figure 13.32.

- Determine the required rip rap layer thickness t (m) to transmit the overtopping flow within the interstitial voids in the riprap using:

$$t = q/(v_i n_p) \tag{13.12}$$

where
v_i = interstitial velocity (m/sec)
n_p = porosity of the riprap

and

$$v_i = 2.48 S^{0.58} C_u^{-2.22} \sqrt{gD_{50}} \tag{13.13}$$

where
S = embankment slope relative to the horizontal (ratio of vertical to horizontal)
C_u = Uniformity coefficient of riprap (D_{60}/D_{10})
g = gravitational constant (9.81 m/sec^2)

If the calculated thickness is greater than the minimum thickness of 2 D_{50} increase the D_{50} or the thickness so all flow is within the interstitial voids. Re-do the calculations until a solution is reached.

Those wanting to use this method should reference the paper which includes a worked example.

The Frizell *et al.* (1998) data was included in the Abt *et al.* (2013) analysis and gave one of the best fits of data at the coarse sizes for which it was developed.

These methods are developed for uniform sized riprap overlying a low permeability base e.g. earthfill, and should not be applied to overtopping of a rockfill dam.

For overtopping of rockfill dams the mechanisms will depend on the zoning of the dam, whether the overtopping occurs during construction of the dam or for the completed dam, and whether the flows will enter into the rockfill or be so large they will flow over the downstream surface. For the former the methods described in Section 10.9.2 can be used.

For overtopping flows empirical data is often used as described in Section 13.10.

13.10 OTHER OVERTOPPING PROTECTION METHODS FOR EMBANKMENT DAMS

ASCE (1994) describes various methods for providing overtopping protection of existing embankment dams. These include;

- RCC and soil cement overlays (PCA 2002)
- Conventional and mass concrete overlays
- Articulated concrete blocks (Frizell and Ruff, 1992; National Concrete Masonry Association, 2010)
- Rockfill filled gabions or mattresses
- Turf reinforcement mats and vegetative cover
- Flow through/reinforced rockfill (Section 13.8)

- Rip rap (Section 13.9)
- Geomembranes, geocells and fabric-formed concrete.

These are speciality topics and the reader is referred to published literature and the involvement of experienced practitioners if these methods are to be used. Wherever these types of "embankment hardening" are considered for use on an embankment, which generally will be for low height structures, the designer should carefully investigate the expected flow conditions, for example, the flow velocities down the face and the likely development of turbulence down the slope and at and along the toe of the protective covering. The inclusion of appropriate filter/drainage layers behind the protective covering should be carefully assessed.

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Chapter 14

Specification and quality control of earthfill and rockfill

14.1 SPECIFICATION OF ROCKFILL

It is common to specify the following for rockfill for dam construction:

a. *Type of rock, degree of weathering and source*
   For example, "granodiorite, slightly weathered to fresh, from Quarry B" or "sandstone, with up to 20% siltstone, slightly weathered to fresh, from spillway excavation".
b. *Layer thickness and maximum particle size*
   The layer thickness varies depending on the rockfill zone, type of dam, and rockfill type, strength and degree of breakdown on rolling Table 14.1 gives some examples.
   The maximum particle size can be the same as the layer thickness. Some e.g. USBR (1991) prefer a maximum particle size 90% of the layer thickness.
c. *Rockfill grading and limits on finer particles*
   The rockfill grading may be further specified depending on the properties required. Ideally rockfill is well graded, so it has a high density, strength and modulus after compaction and has a high permeability. However this is not always

Table 14.1 Examples of rockfill layer thickness.

Zone	*Rock type, strength and breakdown*	*Layer thickness after compaction (m)*
3A	Igneous or metamorphic, high strength, little or some breakdown on rolling.	0.9 or 1.0
3A	As above, adjacent Zone 2B.[1]	0.45 or 0.5
3A	Metamorphic or sedimentary, or weathered igneous, medium or high strength, considerable breakdown on rolling.	0.45 to 0.6
3A	As above, adjacent Zone 2B.[1]	0.45 to 0.6
3B	Igneous or metamorphic, high strength, little or no breakdown on rolling.	1.5 to 2.0
3B	Metamorphic or sedimentary or weathered igneous, medium or high strength, considerable breakdown on rolling.[2]	0.9 to 1.5

Notes: [1]Thinner layers are to assist Zone 3A act as a filter to Zone 2B.
[2]Thinner layers would apply for concrete face rockfill dams where a high modulus is required or for upstream rockfill, with high fines content, or rock which weakens on saturation leading to collapse settlement.

achievable if the rock available tends to break down on rolling to give a gap-graded material with a high percentage of sand. As well, the rockfill will tend to segregate on placement, with the coarser particles being more predominant in the lower half of the lift and the finer in the top half. Many weakly cemented sandstones or weathered other rocks will do this. Even if the rock can be quarried and placed to meet the requirement of being well graded and with high permeability, care needs to be adopted in specifying such requirements, particularly for small dam projects as:

- It is costly and difficult to carry out the testing to determine the particle size in the embankment. For rock with a maximum size of 1 m, a hole about 4 m to 5 m diameter and to the depth of the layer would have to be dug to obtain a representative sample. Several tonnes of rockfill would be involved.
- The upper part of each layer is likely to be finer than the lower part, because of segregation in placing and breakdown of rockfill under the rolling action. For vertical seepage the permeability of this broken material will dominate. If, however, it is a normal situation where horizontal drainage is most important, such breakdown may not be important, provided the lower part of the layer remains permeable.

In any case the breakdown is a fact of life and the design must accommodate it. In most cases the high horizontal drainage capacity is more than adequate, but where a rockfill zone is specifically required to provide vertical drainage, then the designer may have to take some special precautions with the surface layer breakdown. For most projects, for free draining high permeability rockfill, the authors favour an approach which requires that for the rockfill after compaction:

a. The maximum particle size to be the compacted layer thickness,
b. $\leq$20% or 30% passing 19 mm,
c. $\leq$10% or 15% passing 1.18 mm,
d. $\leq$5% passing 0.075 mm and
e. Water not to pond on the surface of the compacted layer.

These gradations apply to the rockfill below the upper surface of each layer which is usually finer because of breakdown under rolling action.

The grading would be checked by a limited number of tests. Routine acceptance or rejection would be by inspection and only measured if it became necessary from a contractual viewpoint to confirm the percentage of finer particles was excessive.

A guide to excessive finer fraction can be if the fill moves excessively as it is being rolled, or trucks bog on the fill. This also indicates the fill does not have a high permeability.

d. *Placement and spreading*

Usually the rockfill is dumped from trucks on the surface and spread by a bulldozer. In so doing the fines are moved onto the upper part of the layer, which creates a smoother working surface for the truck to place the next layer and the stratified permeability discussed above. Oversize rocks are often pushed into a specified zone (e.g. Zone 3B) in the outer part of the dam. To meet the filter requirements is important to ensure that the contact between Zones 2B and 3A does not have an accumulation of large rocks. To ensure this is achieved it is best to

specify a thinner layer thickness for Zone 3A adjacent Zone 2B as shown in Table 14.1. If the usual thickness is used for Zone 3A almost invariably segregation will occur and large boulders will be placed adjacent Zone 2B even with if the specification says this is not allowed. Where the possibility of having larger particles against the Zone 2B material becomes a concern to the designer, it would be better to specify a separate zone of finer rockfill next to the filter zone. Some dam construction authorities, like the Water Corporation in Western Australia, favour such an approach.

e. *Roller type and number of passes*

It is normal to specify smooth steel drum vibrating rollers for compacting rockfill. The roller is usually specified as having a static mass of between 10 tonnes and 15 tonnes and a centrifugal force not less than 240 kN 'at the maximum frequency permitted by the manufacturer for the continuous operation of the roller.' Alternatively the static weight may be specified as a weight per metre of drum (50–55 kN/m), and a centrifugal force/m of drum (125 kN/m). The frequency may also be specified (e.g. 16–25 Hz). Reference should be made to manufacturers' specifications for rollers.

Roller trials are often specified to determine the number of passes. These are discussed below. It is usual to require at least 4 passes of the roller.

f. *Addition of water*

Water is often added to the rockfill to aid compaction and to weaken the rock-to-rock contact points in rock-types which are weakened by wetting. The water should be added by spraying on to the dumped rockfill before spreading. Commonly the amount of water is specified as a percentage of the volume of the rockfill, e.g. 20%.

If the rock forming the fill is strong, produces few fines and is not greatly weakened by wetting, water may not be needed. Rolling trials can assist in assessing whether watering is needed and the amount of rolling required to achieve good compaction.

g. *Durability requirements*

Rockfill is often required to be 'hard' and 'durable'. The means of measuring this are seldom specified and in many dams such a requirement will be unobtainable, e.g. where siltstone or sandstone is being used, which may breakdown on repeated exposure to wetting and drying. It has been demonstrated (e.g. USBR, 1991 and Chapter 2, Section 2.9.1) such breakdown only occurs on the surface of the rockfill, and is not detrimental to the shear strength or compressibility of the rockfill as a whole, provided the rockfill is well compacted and watered during compaction. Nevertheless, water should be used to control dust wherever necessary.

Hence, in general, there should be no requirements on durability for rockfill. The exception might be for some volcanic and altered granitic rocks which can deteriorate with time (see Sections 2.9.3 and 2.9.4.2). For rip-rap, or the outer layer of rockfill on the downstream slope, durability under wetting and drying is important and should be specified. The type of test will depend on the rock type proposed, but should include testing of the substance strength and assessment of the mineral composition, in particular the proportion of secondary minerals in the rock, and observation of blocks of rock left exposed to the weather.

h. *Selection and placement of rip-rap*

Rip-rap on earth and rockfill dams is usually constructed by pushing the larger rocks from the adjacent rockfill zone to the face of the embankment and finishing the face by carefully positioning rocks with an excavator, sometimes fitted with 'claws', to satisfy rock grading requirements for rip-rap, it will be normal to specify the grading more closely than rockfill, e.g. requiring that at least 50% of the rock should be greater than a certain size. Thin rip-rap layers for earthfill dams may be placed progressively as the embankment is built or on the completed face by dumping and spreading with a bulldozer or, more likely, today with an excavator. The excavator has been proven to be a valuable piece of plant on dam construction; its use on riprap placement, for example, not only does not slow down the job, but allows for a neat, well locked-in face of riprap.

i. *Compacted density or void ratio*

Many designers in the past have also had a requirement for a compacted density for the fill, a common requirement being around 2.1–2.2 tonnes/m^3. Alternatively a void ratio (volume voids/volume solids) of 15–25% was required. The design is seldom sensitive to bulk density and, again for large rockfill, it is difficult and costly to measure density or void ratio because of the large volume which has to be sampled. Hence, it is unlikely (and unnecessary) that *in-situ* density will be checked for smaller dams and relatively few tests will be done even for large dams.

14.2 SPECIFICATION OF EARTHFILL

It is common to specify the following for earthfill for dam construction:

(a) *The source and Unified Soil Classification of the earthfill.*

For example, "clay and sandy clay, medium plasticity from Borrow area A".

(b) *Maximum size and particle size distribution.*

The maximum particle size (of gravel or rock fragments) in the earthfill is limited to ensure compaction is not affected. It will usually be specified as not greater than a size in the range 75–125 mm. More commonly and better a maximum size of 75 mm would be specified because large cobbles up to 125 mm will make compaction more difficult.

The particle size distribution is further defined to:

(i) Ensure there are sufficient silt and clay fines passing 0.075 mm to give the required low permeability for the core. It is normal to require at least 15% passing 0.075 mm. For most clays, sandy clay and clayey sands this will be readily met.

(ii) Be compatible with the grading requirements assumed for the design of the filters. This may be achieved by specifying the particle size grading envelope for the soil (based on laboratory tests from the borrow area, as shown in Figure 9.24). It is important to measure the particle size in the embankment after compaction to allow for breakdown of weak rock particles and mixing of fine and coarse soils from the borrow area. During site investigations, the effect of such breakdowns can be assessed

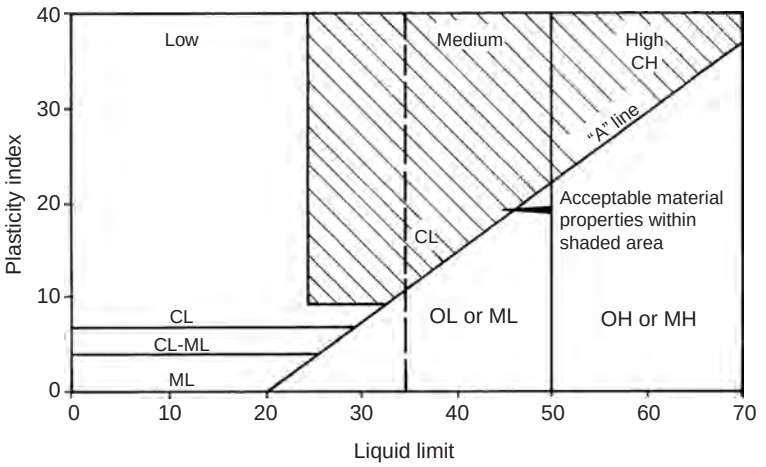


Figure 14.1 An example of specification of Atterberg limits for earthfill.

by carrying out particle size distributions on samples which have been subjected to compaction.

(c) *The Atterberg limits.*

Most authorities specify a minimum plasticity index and some also place a maximum on liquid limit. The latter may be due to the presence in the borrow area of particularly high plasticity clays, which may be difficult to compact, but in general there should be no need to place an upper bound on the liquid limit. In fact there is evidence that more plastic clays are likely to be less erodible than other soils so are advantageous to have in the core.

The authors' preference is to specify an allowable range of liquid limit and plasticity index by relating to the 'A' line on the plasticity chart. Figure 14.1 gives an example. The critical issue really is to specify limits which can be satisfied by the material you wish to use from the borrow area while still giving the material properties the dam is designed for.

The Atterberg limit requirements should be seen only as a way of allowing rejection of unsuitable material if a dispute arises with the contractor. Generally, it should be possible to accept or reject material visually in the borrow area, at least after the job has been under way for some time and the contractor and supervisors are familiar with the materials.

(d) *Density ratio, water content and layer thickness.*

It is common practice to specify a density ratio $\geq 98\%$ of standard maximum dry density, with water content between OWC -1% and OWC $+1\%$, or OWC and OWC $+2\%$, where OWC is standard compaction optimum water content. It should be noted that:

– Standard, not modified compaction should be used. This is to ensure moist compaction which leads to lower permeability, more flexible fills. Compaction at around modified optimum water content leads to high densities,

but the soil structure is likely to be aggregated, leading to a higher permeability and possibly more brittle fill. This is discussed more in Section 6.3.2.5.

- Recent studies (Wan and Fell, 2002, 2003) show dry compaction also is likely to result in clay soils which are more erodible than if they are compacted wet of optimum.
- The requirement for density ratio ≥98% is reasonable and compatible with the water content ranges shown. There is no advantage in specifying a higher density ratio and it may be detrimental in that the contractor will be forced to compact dry of optimum water content. For smaller dams, dams to be constructed in wet climates and soils which are difficult to compact, it would not be unreasonable to relax the compaction requirement to as low as 95% density ratio, provided that compaction is carried out above standard optimum water content. However, compaction to only 95% density ratio at say optimum −3% would lead to a permeable soil structure potentially subject to collapse settlement on saturation forming a concentrated leak and would not normally be acceptable. Figure 14.2 shows some of these effects.

It has been common practice in Australian dam engineering (and elsewhere) to specify a layer thickness of 150 mm after compaction. USBR (1991) suggest 150–200 mm loose thickness for rolling with sheepsfoot rollers, and 225–300 mm loose thickness for rolling with 50 tonne rubber typed rollers.

The 150 mm layer thickness after compaction is probably unnecessarily thin for modern heavy rolling equipment. Table 14.2 from Forssblad (1981) shows for different soil types the practical maximum layer thicknesses for a range of roller types and weights indicating that 6 tonne to 10 tonne pad foot rollers can compact layers up to 0.3 m and 0.4 m thick respectively. While these figures may represent an upper practical limit, they do show that a 150 mm layer can be unnecessarily restrictive. It could be argued that, as a performance specification has been adopted for earthfill, i.e. water content and density ratio, there is no need for the 'method or procedure specification' limitation on layer thickness. The authors' own preference is to specify layer thickness but to use more realistic figures, e.g. 200 mm or 250 mm after compaction for a 11 tonne self propelled pad foot roller. Trials should be done with the equipment that is to be used to prove that the proposed layer thickness will be acceptable.

(e) *Placement and spreading.*

Earthfill may be placed by scrapers or dumping from a truck and spreading with a grader or bulldozer. Oversize material should be removed before compaction.

The surface of the previously compacted layer should be scarified prior to placing the next layer of fill to ensure good bond and to remove any cracks which may have formed on the surface between placing layers. It may be necessary to add a small amount of water to the scarified surface prior to placing the fill. Care should be taken to ensure a uniform material. Segregation is likely to be a problem, if gravel sized particles are allowed in the earthfill.

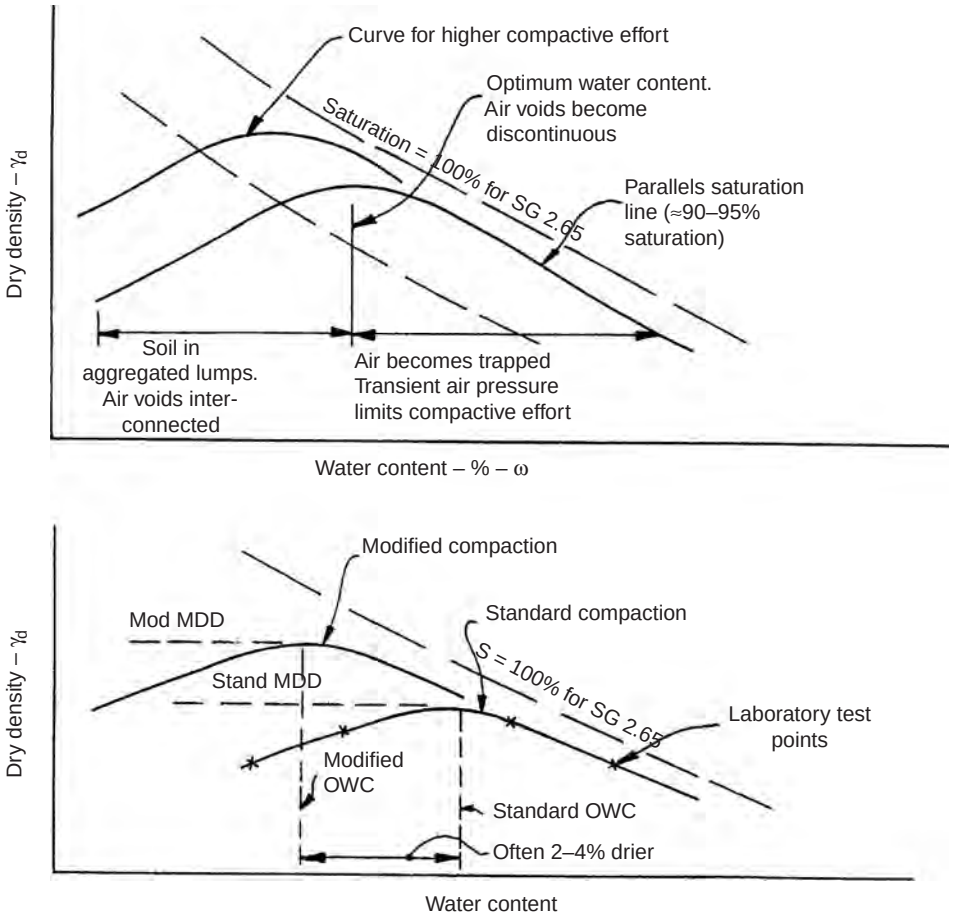


Figure 14.2 Compaction water content and the effect on properties.

(f) *Roller type, weight, and number of passes.*

It is normal to specify the roller type and weight. For earthfill it is most common to require a tamping foot (or 'sheep's foot' or 'pad foot') type roller because:

- The tamping foot action breaks up pieces of cemented soil or weathered rock in the fill.
- The rough surface of the layer left by a tamping foot layer allows better bond between layers.
- There is less of a tendency for formation of large shear surfaces beneath the roller when a tamping foot roller is used. This tendency is a particular problem in high plasticity, fine grained soils placed wet of optimum.
- The compaction water content is less critical than for rubber tyred rollers, i.e. the shape of the dry density-water content curve is broader for tamping feet type rollers.

Table 14.2 Practical maximum layer thickness (m) after compaction for different types of rollers at different applications (Forssblad 1981).

Roller type static weight (drum module weights in brackets)	*Embankment*					
	Rockfill[(1)]	*Sand gravel*	*Silt*	*Clay*	*Sub base*	*Base*
Towed vibr. rollers						
6 tonne	0.75	*0.60	*0.45	0.25	*0.40	*0.30
10 tonne	*1.50	*1.00	*0.70	*0.35	*0.60	*0.40
15 tonne	*2.00	*1.50	*1.00	*0.50	*0.80	–
6 tonne padfoot	–	0.60	*0.45	*0.30	0.40	–
10 tonne padfoot	–	1.00	*0.70	*0.40	*0.60	–
Self-propelled vibr. rollers						
7(3) tonne	–	*0.40	*0.30	0.15	*0.30	*0.25
10(5) tonne	*1.50	*1.00	*0.70	*0.50	*0.40	
8(4) tonne padfoot	–	0.40	*0.30	*0.20	0.30	–
11(7) tonne padfoot	–	0.60	*0.40	*0.30	0.40	–
15(10) tonne padfoot	–	1.00	*0.70	*0.40	0.60	–
Vibr. tandem rollers						
2 tonne	–	0.30	0.20	0.10	0.20	*0.15
7 tonne	–	*0.40	0.30	0.15	*0.30	*0.25
10 tonne	–	*0.50	*0.35	0.20	*0.40	*0.30
13 tonne	–	*0.60	*0.45	0.25	*0.45	*0.35
18 tonne padfoot	–	0.90	*0.70	*0.40	0.60	–

Notes: (1) For rock fill only rollers especially designed for this purpose.
(2) Most suitable applications marked*.

The weight is best specified as a weight per metre length of roller, e.g. 6 tonnes/metre length as this better defines the contact pressure than total weight. Some authorities also specify the geometry and arrangement of the 'feet' on the roller. This would seem to be unnecessarily restrictive for most dam construction. Sheepsfoot rollers are not readily available now and are not as popular as they require more energy to pull them over the fill.

Sherard *et al.* (1963) report rolling trials carried out at the US Army Experiment Station, which showed that the foot size did affect the maximum dry density and optimum water content (as did the tyre pressure for rubber tyred rollers). Again, it could be argued that, provided the performance specification is defined, there is no need to also specify roller weight and type.

There are occasions where Zone 1 earthfill is better compacted with different roller types, e.g.:

– Where water contents are naturally high and compaction is required at well above optimum water content. In these cases a rubber tyred roller may be preferable as it will be less likely to become clogged with the wet, sticky clay. Knight *et al.* (1982) indicates that Swamp dozers with low ground pressure were used to compact very high water content (80–120%) halloysite clays for the Monasavu dam in Fiji. Swamp dozer or similar techniques lead to

compressible cores with high pore pressure, but they perform the primary function of low permeability. Sherard *et al.* (1964) and Kjaernsli *et al.* (1992) describe similar techniques used for wet, silty, sandy gravel glacial soils in Sweden and Norway. Penman (1983) describes other dams where materials with high water content have been placed.

– Where the fill is a weathered rock which breaks down to clayey silty sand/silty sand, steel drum rollers, compacting in thin layers (150 mm) may give better breakdown of the rock and compact it more readily. Best performance in these cases may be obtained without vibration.

The number of passes is best determined from field roller trials at the beginning of construction. Typically 6 to 8 passes is sufficient. Very little will be achieved by more than 8 passes and if more passes are required it is almost certain that control of layer thickness has been lost, or moisture conditioning is inadequate and should be rectified.

(g) *Water content adjustment.*

Most specifications correctly require that water content adjustment be carried out in the borrow area, with only minor adjustment allowed on the embankment. This means that dry (or wet) soil in the borrow area is brought to a uniform water content by irrigating, harrowing, watering (or drying), and reworking before transportation to the embankment. It is impractical to do this on the embankment. Soils which are particularly dry or wet of the required water content may have to be conditioned for some days or even weeks before use in the embankment. Failure to do this is the common cause of difficulty in achieving specified compaction requirement. The authors have seen several dams where failure rates for compaction have been very high because the contractor has refused to allow sufficient time for soils to condition to the correct moisture content. Generally, once a working stockpile of "cured" material was prepared then the test pass rate improved markedly.

Sherard *et al.* (1963) give details of procedures which may be necessary to raise the water content of dry soils in the borrow area. They indicate that:

– Irrigation may be achieved by ponding of water or spray irrigation. Ponding is only suited to flat areas and can result in large evaporation losses.
– The water content seldom becomes too high – most soils only take in water up to about the optimum.
– Ripping to 0.6 m to 0.9 m will assist in allowing water to penetrate. Contour ploughing can assist in hilly borrow areas.
– Water conditioning up to 4.5 m depth has been successfully achieved.

Earthfill which dries out, or gets wet, on the embankment must be tyned, its moisture adjusted, then thoroughly reworked with a grader before recompaction. If the soil is judged too wet or dry to be adjusted on the embankment, it should be removed from the embankment.

The risk of wetting earthfill on the embankment if rain is about to fall is usually reduced by requiring the surface to be 'sealed' with a smooth drum roller, and contoured to allow runoff of surface water. The sealed surface must be tyned prior to placing the next layer.

(h) *Fill adjacent to the foundation*

It is normal to specify that the earthfill which is to be placed adjacent (within say 0.6 m) of the dam foundation on rock is to be composed of finer, more plastic soil available from the borrow area and is to be compacted at a higher water content (e.g. OWC + 2% or OWC + 3%) with rubber tyred construction equipment or rollers. This is to facilitate squeezing the soil into the irregularities in the foundation. Generally there is no 'performance' specification (i.e. density ratio), because the layers are thin and testing is impracticable. On Thomson Dam, Snowy Mountains Engineering Corporation (SMEC), designing for the Melbourne and Metropolitan Board of Works (MMBW), required that the contact zone be compacted to a density 100% of that which would be achieved by compacting in the laboratory with standard compactive effort, at the field water content. This is a reasonable approach which allows for the high compaction water content, but should not be specified except for such large dams as Thomson (160 m high). For most dams, a methods specification with good water content control is sufficient.

(i) *Compaction of the edges of fill*

Under normal operations, the outer 1 m to 1.5 m (measured horizontally) of an earthfill embankment will not be adequately compacted by rollers. It is necessary to specify either, that the embankment is constructed oversize and trimmed back to the required lines by removing this poorly compacted soil, or to require rolling of the surface up and down the slope.

If left in place, the poorly compacted soil will often soften and lead to surficial sliding, particularly on steeper slopes. While this will not in itself lead to failure of the embankment, subsequent erosion or sliding may cause problems.

14.3 SPECIFICATION OF FILTERS

It is common to specify the following for filters:

(a) *Type of material and source*

For example: "washed, graded sand and gravel alluvium from Borrow Area F" or "crushed, graded, slightly weathered to fresh granodiorite from Quarry G".

(b) *Particle size grading*

This is specified as outlined in Section 9.4.1.

It is important to make it clear in the specification for filters, that the particle size specification will be strictly adhered to, including the limitations on fines (passing 0.075 mm) content, and that the contractor should expect to have to wash, screen into stock piles of individual sizes and combine to obtain the required grading. It is unlikely that the required grading for fine filters in particular can be obtained directly from the borrow pit or by crushing alone although some engineers do think such can be readily done.

The gradings should be specified as in place in the embankment unless trials or experience have shown that the material does not break down during compaction. The testing should however be done in stockpiles as well as in the

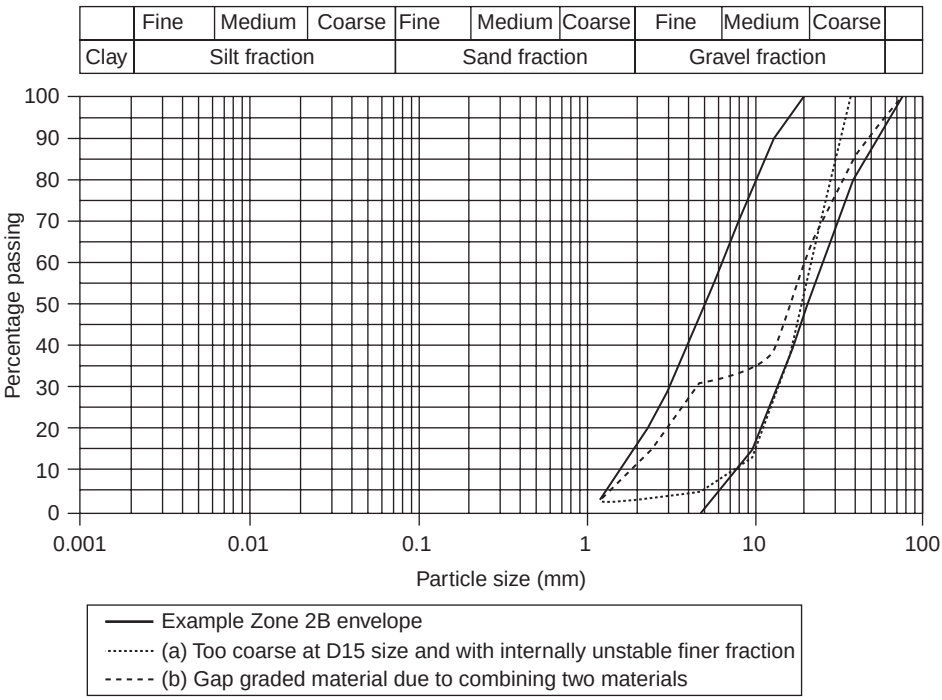


Figure 14.3 Examples of Zone 2B: (a) Too coarse at D15 size and with internally unstable finer fraction; (b) Gap graded.

embankment as it is better to identify-out-of specification materials ahead of placing them into the embankment.

The author's experience is that many, or even most filter suppliers and contractors have problems early in projects meeting the specified particle size distributions:

- Fines contents for Zone 2A are often higher than specified because of inadequate washing in the processing plant, inability to combine the product with the right quantities of specific size ranges, breakdown on compaction, and contamination in stockpiles.
- Zone 2B gradations often are shaped like those in Figure 14.3(a) with the material either too coarse or just inside the coarse limit for the 15% passing, with a flat "tail" of finer material. These are often internally unstable and if the finer fraction washes out the filter will fall further outside the specified limits. This is particularly a problem for filters manufactured by crushing rock in a single operation without screening and re-combining the product.
- Zone 2B being gap graded as shown in Figure 14.3(b) is a result of the filter being made by combining two materials, neither of which have the mid size

range. This is often the coarse sand. These materials will also be internally unstable and may fall outside specified limits if suffusion occurs.

One way of reducing the impact of these issues on a project is to require tenderers to supply samples of the filters they propose to use during the tender period and/or to require filter manufacture or supply to be done well ahead of placement in the embankment so the problems can be sorted out in time to not impact on the overall schedule. Field placement trials should be required early in the contract to determine if breakdown occurs during compaction.

(c) *Durability*
This is discussed in Section 9.4.2.

(d) *Roller type, weight, number of passes, and required density index*
These are discussed in Section 9.5.3.

(e) *Dimensions and sequence of placement*
These are discussed in Sections 9.5.1 and 9.5.2.

(f) *Control of quality in stockpiles*
It is important that the specifications call for good practice in handling the filters in stockpiles. This includes avoiding segregation, and contamination.

14.4 QUALITY CONTROL

14.4.1 General

The question of quality control for earth, earth and rockfill and rockfill dams was addressed by ICOLD and reported in Bulletin 56 (ICOLD, 1986b). The following discussion summarises the concepts in that bulletin, supplemented by some opinions of the authors and data from Australian practice.

14.4.2 'Methods' and 'performance' criteria

There are two principal types of technical specifications:

(a) Method or procedure specifications which describe how the construction is to be carried out, in order to achieve the desired end product. The specifications stipulate to the contractor the materials to be used, the equipment and construction procedures, e.g. specification for earthfill includes:

- the source of materials
- the water content
- layer thickness
- roller type and weight
- number of passes.

This places the onus on the owner/engineer to have established procedures which will yield a satisfactory product.

(b) Performance or end product specifications which describe the end result to be achieved by the contractor. It is the contractor's responsibility to select materials,

equipment and methods to obtain the specified end product, e.g. specification for earthfill includes:

- Particle size gradation
- Atterberg limits
- Water content
- Density ratio.

Many dam specifications are a mixture of these two alternatives. This is often unnecessary and can lead to inefficient construction procedures (e.g. requiring too thin a layer for compacting earthfill), unnecessary costly testing (as in requiring particle size and density of rockfill) and disputation between constructor and owner/engineers when the 'methods' part of the specification fails to produce the required performance criteria.

In most projects, methods specifications become the routine quality control technique, even if a performance specification has been used for the contract documents. This is the only practical way of allowing work to proceed without unnecessary delay.

Method specifications are the only practical way to specify the designers' requirements for compaction of rockfill, and coarse (Zone 2B) filters. These materials are too coarse to carry out laboratory tests to get maximum (and minimum) densities for comparison with the field densities because of the large samples which would be required.

On some projects the owner/engineer will be responsible for the quality control, i.e. the inspection, testing and assessing whether the required quality standard has been achieved and will have the authority to require additional work to achieve the standard or to reject the non-conforming work. The owner/engineer in these projects will also be responsible for quality assurance, i.e. that the quality control standards are valid for the dam, that the specified tests are being implemented and correctly performed, that the quality control plan is working and that records and reports are verified and maintained.

On other projects the quality control aspect may be incorporated into the contract and be the contractor's responsibility. This can be quite satisfactory provided that the contract documents clearly define the level of inspection and testing required and the standards to be adopted and that the inspection and testing group are given authority by the contractor's management to enforce the standards. Often the testing will be subcontracted to a specialist geotechnical consultant. This assists in maintaining the independence of the inspection and testing group from the main contracting staff.

Whichever method is adopted, testing and QA procedures cannot replace experience visual inspection and commonsense changes to specifications, methods and control when they are needed.

14.4.3 Quality control

(a) *Inspection*

Inspection must always form a critical part of a quality control plan. The field and laboratory testing program should be seen first as establishing the methods

required to achieve the required quality, then ensuring that the quality is being maintained and as a definitive, quantifiable means of rejecting substandard work. It is clearly impractical to test the whole of the completed product so one must rely on visual inspection to maintain overall quality.

It is important that the inspectors are properly trained and briefed on the implications of substandard work. It is also important to recognise that inspectors will often be needed in the borrow areas, as well as on the embankment, so that unsuitable material can be rejected before it reaches the embankment. Some practical guidance for field inspectors is given in USBR Design Standard No. 13 Chapter 10 Embankment Construction (USBR 2012).

(b) *Testing*

Most testing which is carried out is for quality control, i.e. to ensure that the requirements of the specification are being met. The selection of areas or materials for testing may be done in either of two ways:

- Selecting those areas which are judged by the supervisor/inspector to be least likely to meet the specification. This can assist in reducing the quantity of testing and, if the testing shows acceptable performance, should ensure the overall adequacy of the construction work. It does, however, require independent and experienced supervisors.
- Selecting test areas at random, at the minimum recommended frequency. This will yield test results which better reflect the overall condition of the construction work, as the biased sampling of the first alternative is avoided.

The latter method is better suited to establishing statistical limits to the testing, allowing recognition of the fact that there is a statistical sampling error and that, within a large mass of earth and rockfill, the failure of a small proportion of the material to meet the basic specification criteria will not affect overall performance (provided the failures are not representative of poor compaction for example adjacent a conduit or wall).

Which of these two methods is adopted is also related to the specification limits used, and whether the supervisors are given any latitude in accepting material which falls below the specification.

For example, if earthfill is specified to have a density ratio in excess of 98%, with a water content optimum minus 1% to optimum plus 1%, it would be reasonable in most cases to accept an area which has tested at 97% density ratio, OWC + 2%, as this will still yield a low permeability fill. However, one would almost certainly reject an area which has tested at 95% density ratio, at OWC −3%, as this will give a more permeable, erodible fill.

Similarly, if a relatively low compaction standard is set at (say) 95% density ratio, no 'failures' should be accepted.

The authors' opinion is that it is desirable to establish such guidelines in the design stage of a project and to detail them in the specification either in a descriptive or a statistical form. Alternatively, a rigid specification limit may be set, but the site supervision staff should be advised as to the degree of flexibility which can be used in applying the specification. This may be incorporated into the specification by saying for example that 9 in 10 tests in a running sequence must

Table 14.3 Suggested minimum required frequency of construction testing – Zone 1 earthfill and Zone 1A random earthfill.

Zone	*Fill volume m^3*	*Particle size distribution (2, 3)*	*Atterberg limits (2, 3)*	*Permeability in situ (2, 3)*	*Dispersivity (2, 3)*	*Water content (4)*	*Density ratio (4)*
Zone 1	<100,000	500 m^3 to 2,000 m^3	4,000 m^3 to 10,000 m^3	5,000 m^3 to 20,000 m^3	4,000 m^3 to 10,000 m^3	Each layer, or 100 m^3 to 500 m^3	Each layer, or 100 m^3 to 500 m^3
	100,000 to 1,000,000	2000 m^3 to 5,000 m^3	4,000 m^3 to 10,000 m^3	10,000 m^3 to 40,000 m^3	4,000 m^3 to 10,000 m^3	Each layer, or 250 m^3 to 1000 m^3	Each layer, or 250 m^3 to 1000 m^3
	>1,000,000	2000 m^3 to 5,000 m^3	4,000 m^3 to 20,000 m^3	40,000 m^3 to 100,000 m^3	4,000 m^3 to 20,000 m^3	Each layer, or 500 m^3 to 2,000 m^3	Each layer, or 500 m^3 to 2,000 m^3
Zone 1A	<100,000	2,000 m^3 to 10,000 m^3	2,000 m^3 to 10,000 m^3	5,000 m^3 to 20,000 m^3	2,000 m^3 to 10,000 m^3	Each layer, or 250 m^3 to 1,000 m^3	Each layer, or 250 m^3 to 1,000 m^3
	100,000 to 1,000,000	5,000 m^3 to 50,000 m^3	5,000 m^3 to 50,000 m^3	20,000 m^3 to 100,000 m^3	5,000 m^3 to 50,000 m^3	Each layer, or 500 m^3 to 2000 m^3	Each layer, or 500 m^3 to 2000 m^3
	>1,000,000	25,000 m^3 to 100,000 m^3	25,000 m^3 to 100,000 m^3	50,000 m^3 to 200,000 m^3	25,000 m^3 to 100,000 m^3	Each layer, or 500 m^3 to 2,000 m^3	Each layer, or 500 m^3 to 2,000 m^3

Notes. (1) Lower volumes to apply to smaller volumes and more variable soils.
(2) Zone 1;A minimum of 10 tests for each property for uniform soils, 20 to 50 tests for variable soils.
(3) Zone 1A;A minimum of 5 tests for each property for uniform soils, 10 to 20 tests for variable soils.
(4) Whichever gives the greater number of tests.

have a density ratio in excess of 98%, and that up to 1 in 10 tests may be as low as 97% density ratio.

The question of how many tests should be carried out on earth and rockfill is virtually impossible to answer, as it is interrelated to the specification standards, the competence of the contractor and the inspectors and the site conditions (e.g. variability of materials, climate).

Tables 14.3, 14.4 and 14.5 give suggested minimum frequencies of testing for earthfill, rockfill and filters. These are based on ICOLD (1986b), Australian specifications, and the authors' experience.

Table 14.4 Suggested minimum required frequency of construction testing – Zone 3A and 3B rockfill.

Zone	*Fill volume* m^3	*Particle size distribution (2, 3)*	*Density in-situ (2, 3)*	*Permeability In-situ (2, 3)*	*Petrographic analysis*
Zone 3A	<100,000	10,000 m^3 to 20,000 m^3	25,000 m^3 to 50,000 m^3	10,000 m^3 to 20,000 m^3	See Note (4)
	100,000 to 1,000,000	20,000 m^3 to 100,000 m^3	50,000 m^3 to 200,000 m^3	20,000 m^3 to 100,000 m^3	See Note (4)
	>1,000,000	100,000 m^3 to 500,000 m^3	200,000 m^3 to 1,000,000 m^3	100,000 m^3 to 500,000 m^3	See Note (4)
Zone 3B	<100,000	10,000 m^3 to 20,000 m^3	25,000 m^3 to 50,000 m^3	10,000 m^3 to 20,000 m^3	See Note (4)
	100,000 to 1,000,000	20,000 m^3 to 100,000 m^3	50,000 m^3 to 200,000 m^3	20,000 m^3 to 100,000 m^3	See Note (4)
	>1,000,000	100,000 m^3 to 500,000 m^3	200,000 m^3 to 1,000,000 m^3	100,000 m^3 to 500,000 m^3	See Note (4)

Notes.
(1) Lower volumes to apply to smaller volumes and more variable sources of rockfill.
(2) Zone 3A; A minimum of 5 tests for each property for uniform rockfill, 10 tests for variable rockfill.
(3) Zone 3B; A minimum of 5 tests for each property for uniform rockfill, 10 tests for variable rockfill
(4) Carry out sufficient tests to define petrography. Number will depend on the variability of materials in the quarry. Do a minimum of 3 tests for Zone 3A and Zone 3B.

(c) *Reporting*. It is important that a complete record should be kept of all construction operations. These are invaluable in the event that repairs or modifications are required, if the embankment is to be raised in the future and for surveillance during the life of the dam. Records are also important in respect to contractual and insurance claims. The reporting should include:

- Plans and specifications, including amendments and work as constructed.
- Final construction report written by the resident engineer.
- Monthly progress reports and reports on technical meetings.
- Reports from dam review panel if one is appointed.
- Laboratory test reports, including clear definition of location and level of samples tested and differentiating between original tests and retests after failures.
- Daily reports by all supervisory personnel and inspectors in the form of diaries. These should concern adequacy of progress and comments on decisions.
- Photographs taken on a regular basis to show placement and compaction of earthfill, filters and rockfill.
- Drawings showing the compilation of the geological mapping of the foundations.
- Record of any design changes made during construction.

Table 14.5 Suggested minimum required frequency of construction testing – Zone 2A and 2B filters.

Zone	*Fill volume m^3*	*Particle size distribution (1, 8)*	*Relative density*	*Permeability In situ (3)*	*Soundness tests (4)*	*Particle shape (5)*	*Petrographic analysis*
Zone 2A	<10,000	200 m^3 to 1,000 m^3	See note (2)	500 m^3 to 2,000 m^3	1,000 m^3 to 2,000 m^3	1,000 m^3 to 2,000 m^3	See Note (6)
	10,000 to 100,000	200 m^3 to 1,000 m^3	See note (2)	1,000 m^3 to 4,000 m^3	2,000 m^3 to 10,000 m^3	2,000 m^3 to 10,000 m^3	See Note (6)
	>100,000	200 m^3 to 1,000 m^3	See note (2)	4,000 m^3 to 10,000 m^3	4,000 m^3 to 20,000 m^3	4,000 m^3 to 20,000 m^3	See Note (6)
Zone 2B	<10,000	200 m^3 to 1,000 m^3	N/A	500 m^3 to 2,000 m^3	1,000 m^3 to 2,000 m^3	1,000 m^3 to 2,000 m^3	See Note (6)
	10,000 to 100,000	200 m^3 to 1,000 m^3	N/A	2,000 m^3 to 10,000 m^3	2,000 m^3 to 10,000 m^3	2,000 m^3 to 10,000 m^3	See Note (6)
	>100,000	200 m^3 to 1,000 m^3	N/A	5,000 m^3 to 20,000 m^3	4,000 m^3 to 20,000 m^3	4,000 m^3 to 20,000 m^3	See Note (6)

Notes.
(1) Zone 2A; A minimum of 20 tests for uniform quality of supplied filters, 20 to 100 tests for variable quality of supplied filters. Test frequency may be reduced after the early stages of construction if quality warrants.
(2) Zone 2A; Sufficient tests to establish method specification for compaction. Zone 2B not applicable.
(3) Amount of testing depends on how critical permeability is to performance of the dam.
(4) Sodium sulphate soundness; UCS; specific gravity; absorption; wet/dry strength; Minimum 3 tests for each source.
(5) Flakiness index; misshapen particles.
(6) Carry out sufficient tests to define petrography. Number will depend on the variability of materials in the quarry. Do a minimum of 3 tests for Zone 2B.
(7) Lower volumes to apply to smaller volumes and more variable filter sources.
(8) Particle size distributions should be taken from stock piles and after compaction in the embankment.

The authors have been involved in a major dam foundation failure where daily reports and photographs proved invaluable. More often this data has proven invaluable for assessing the safety of existing dams either in a deterministic or risk based framework.

14.4.4 Influence of non technical factors on the quality of embankment dams

In the Casagrande volume (Hirschfield and Poulos, 1972), Professor Ralph B. Peck highlighted some of the "facts of life" relating to the influence of non technical factors on construction of embankment dams. Professor Peck points out that many shortcomings in dam engineering relate to the attitudes and actions of the owner, designer, contractor and technical consultants and these can outweigh the real technical issues.

His comments are still relevant today and are recommended reading for all who are involved in dam engineering. Milligan (2003) also covers some of these issues.

14.5 TESTING OF ROCKFILL

14.5.1 Particle size, density and permeability

The testing of rockfill to determine the particle size compacted density and permeability is complicated by the large size of the rock particles.

Bertram (1972) describes field density tests in rockfill with a maximum size of 0.45 m, using a 1.8 m diameter density ring to define the density-in-place hole. The hole was dug to the full depth of the layer. ICOLD (1986b) give details of testing for Oroville Dam, where a 1.8 m diameter ring was used for rockfill up to 0.6 m size. Hence, a ring size 3 to 4 times the maximum particle size has been used to get representative samples. For equivalent samples of rock up to 1 metre maximum size, a ring and hole 3 metres diameter would be needed, giving about 15 tonnes of rockfill to be excavated for a 1 metre thick layer.

Test gradings on rip-rap are even more difficult to do. Spreading of a sample of rip-rap over the ground and individual measurement of rock particles within a defined strip has been used. On Prospect dam remedial works (Sydney, 1996), one supplier went so far as weighing the individual particles.

Permeability of rockfill can be determined by ring infiltration tests *in situ*. If the rockfill is permeable the quantities of water involved would be huge and the results of doubtful value. For low permeability rockfill, tests through say 1.0 m to 1.8 m diameter rings, depending on particle size, could be carried out. In most cases a subjective assessment, by observing whether water will pond on the surface or in a hole dug through a layer, will be adequate to ascertain if the fill is "free draining" or not. Bertram (1972) and ICOLD (1986b) give references for field testing of rockfill.

The particle size distribution and permeability of rockfill are important properties particularly for existing dams where filters and transitions may not satisfy modern no-erosion filter criteria because the gradation of the rockfill may become a critical in providing some degree of filtering if it is sufficiently fine to satisfy some or excessive erosion criteria. The permeability of the rockfill will also be critical to assessing if the rockfill has sufficient discharge capacity to prevent unravelling or slope instability under the flows which might occur before the rockfill "filter" seals.

Because of this it is sometimes necessary to do these tests in existing dams, and it is good practise to do some tests in new dams.

14.5.2 Field rolling trials

Field rolling trials should be carried out:

a) During site investigations, to ascertain the degree of breakdown of rock under rollers, the resulting density and particle size distribution, permeability and modulus of compressibility.

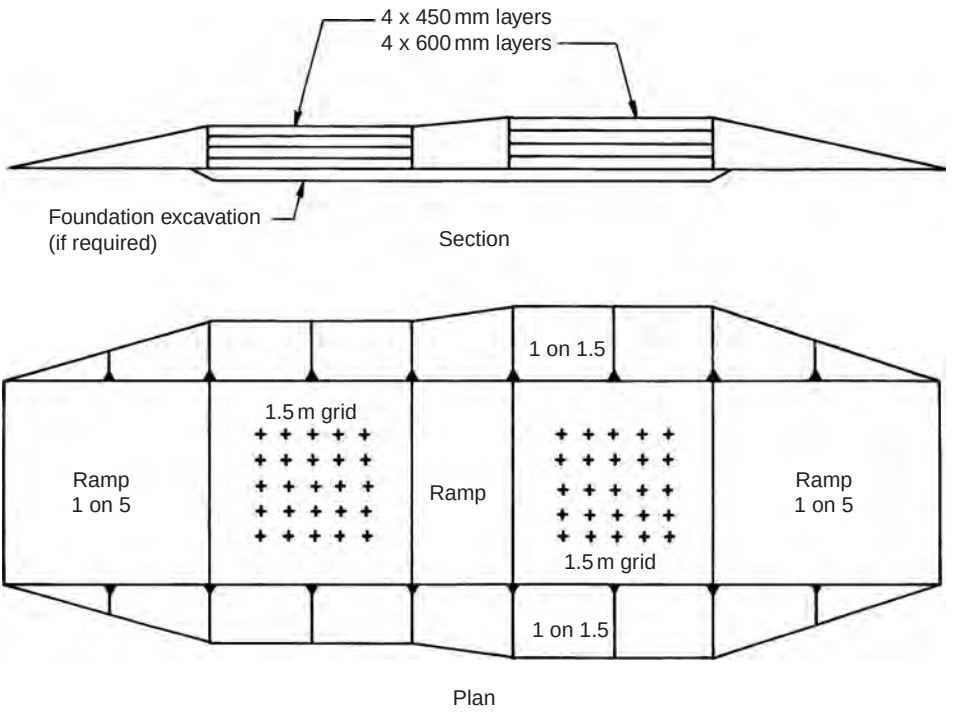


Figure 14.4 Layout of rockfill roller trial (Bertram, 1972).

b) At the beginning of construction, to determine the number of passes of the roller or rollers being used for construction to required to achieve the required degree of compaction.

Figure 14.4 from Bertram (1972) details a layout for roller trials on rockfill.

Bertram makes the following points:

1. An area sufficient to give 25–30 measuring points is necessary to overcome non-uniformity in the rockfill.
2. The grids can be set out as shown (1.5 m) or 1.2 m × 1.3 m or 1.5 m × 2.1 m.
3. Measurements should not be taken less than 3 m from the edge.
4. Several layers (4 to 5 minimum) are required.

For construction, the specification usually requires that trials be carried out on the embankment, requiring an area of about 200 m^2 for each of the test areas.

In the trials, the settlement of the surface of the rockfill is measured after each pass of the roller and the results plotted as average settlement (or % settlement) vs number of passes. Figures 14.5 and 14.6 show the results of roller trials at Murchison Dam, and at Boondooma Dam.

The Boondooma Dam trials show the effect of watering the rockfill.

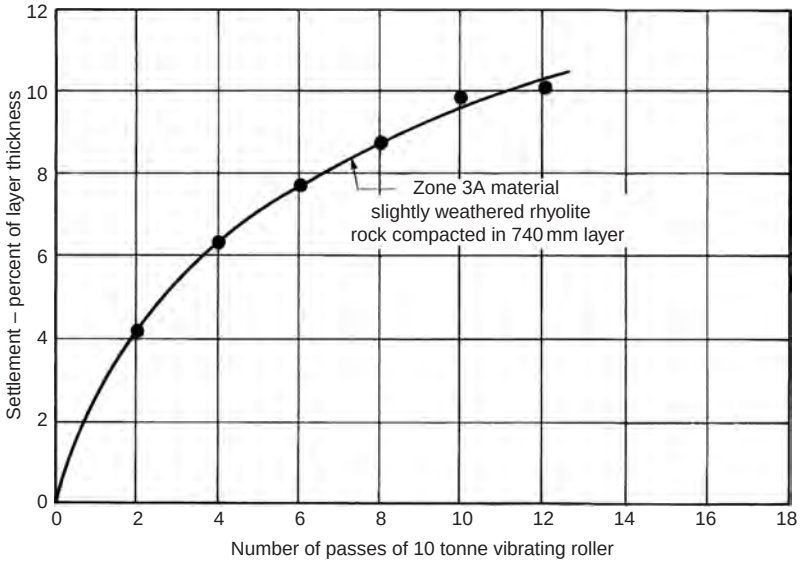


Figure 14.5 Murchison Dam rockfill roller trials (Fitzpatrick *et al.*, 1985).

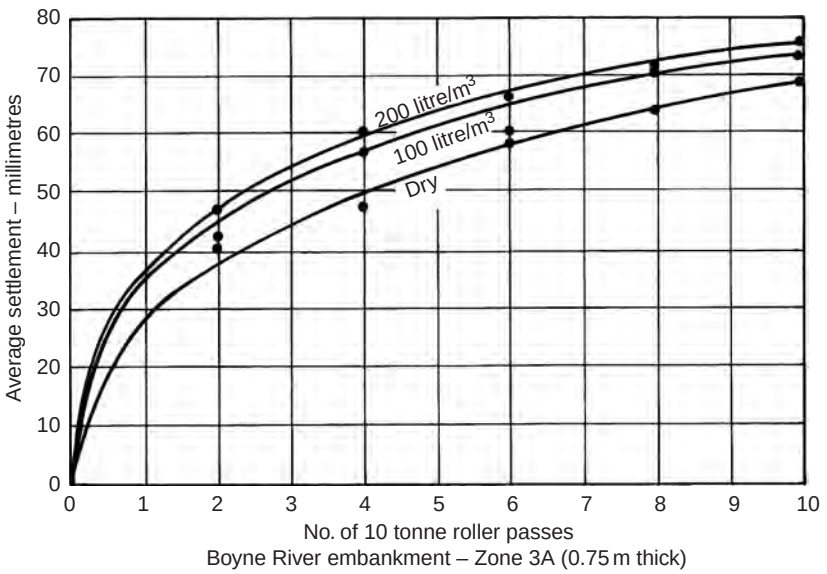


Figure 14.6 Boondooma Dam rockfill roller trials (Rogers, 1985).

It can be seen that the additional compaction achieved after, say, 4 or 6 passes is relatively small. Since this additional compaction is often being achieved largely by breakdown of the upper part of the layer, it is common to limit the number of passes to 4 or 6, seldom more than 8.

A key factor is any trial rolling during site investigations is for equipment representative of what will be used on the dam to be used in the trial. Too often, for example, the equipment is too small or not enough water can be got to the trial rolling site, so the benefits of the trial are lost.

For trials at the beginning of construction the rollers the contractor proposes to use for construction should be used.

It is also useful to excavate exploratory trenches across the trial pad after the compaction trials have been completed to observe the structure within the rockfill layers.

14.6 TESTING OF EARTHFILL

14.6.1 Compaction-test methods

The degree of compaction of cohesive earthfill is determined by the density ratio, where

$$\text{Density ratio} = \frac{\text{dry density in place}}{\text{maximum dry density}}$$

For earthfill in dams, the maximum dry density should be obtained using the standard compaction method (also known as standard Proctor) using the standard applied in the country, e.g.:

AS1289 5.1.1 – Australia
ASTM D698USA
BS1377 4.1 Test 12 – United Kingdom.

The *in situ* density may be determined using the sand replacement, rubber balloon or core cutter method:

AS1289 5.3.1, 5.3.2 and 5.3.3
ASTM D1556, D2167 and D2937
BS1377 4.1.

If the density and *in situ* water content is determined by nuclear density methods:

AS 1289.5.8.1
ASTM D6938-10
BS 1377 Part 9.

For cohesionless soil (sand, silt, sand/gravel) the degree of compaction is determined by density index, (or relative density), where

$$\text{Density index} = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$$

where e = void ratio in place

e_{max} = void ratio in loosest state
e_{min} = void ratio in most compact state

$$\text{Also density index} = \frac{\gamma_{d\,max}\,(\gamma_d - \gamma_{d\,min})}{\gamma_d(\gamma_{d\,max} - \gamma_{d\,min})} \times 100\% \tag{14.1}$$

where γ_d = dry density in place
γ_{dmax} = dry density in most compact state
γ_{dmin} = dry density in loosest state.

The maximum and minimum dry densities are determined in the laboratory using the standard applying to the country, e.g.:

- AS1289 5.5.1
- ASTM D2049
- BS1377 4.3.

These methods all require the water content to be determined using an oven. As this usually takes about 24 hours at 110°C it may be unacceptably long for the construction condition.

For this reason it is common to specify that routine quality control will be carried out using the Hilf method which is described in detail in USBR (1985) and is a standard test (AS1289 5.7.1). The method allows approximate determination of the density ratio within 1 hour of the density in place test, and accurate determination the next day when the water content is confirmed. The Hilf maximum density and OWC should be checked against the corresponding values from the standard laboratory compaction test to see if there are any consistent differences in the test results. Such differences can lead inadvertently to fill being placed that is either too wet or too dry when compared to what the designer intended. Some specifications allow for disputes to be settled based on the measured water content.

Other methods which can be used to allow a more rapid determination of water content and, hence, density ratio, include drying the soil by:

- Microwave oven
- Methylated spirits
- High temperature oven
- Heating the soil on a hot plate with gas burners.

These methods are more approximate, and require calibration between them and water contents determined in the standard oven technique, to allow for adsorbed water being driven off by high temperature drying.

The more commonly used method of routine compaction control is the use of nuclear methods for determination of density and water content. In this method gamma rays are used to determine the density and neutrons to determine the water content. This may be done by direct transmission, backscatter or less commonly the air gap methods. The most common method is backscatter, because of the difficulty of penetrating the source into the compacted fill, particularly if the fill has gravel particles in it.

The method requires calibration against standard blocks before the equipment is used. Ideally it should be calibrated against compacted materials of known density and

water content (obtained by sand replacement methods) from the site. In practice now this is seldom done. This is acceptable provided the earthfill is not too variable or has too high gravel content.

14.6.2 Compaction control – some common problems

Some common problems which arise in compaction control and which lead to disputation between contractor and engineer include:

a) *Specifying too high a compaction standard*, e.g. 100% density ratio, standard compaction or 98% density ratio, modified compaction, for clay soils, or 100% density index for granular soils. These are virtually unobtainable, even with very heavy rolling equipment and, as discussed in Section 14.2, is undesirable for dam construction because it can only be achieved by compacting the clay fill dry, resulting in a, permeable fill potentially subject to collapse settlement, or by over-compacting granular filter materials, giving excessive breakdown and generation of fines
b) *Specifying unnecessarily restrictive water content range*. Specification limits must be realistic to match the available materials. If the soil in the borrow area is, say, 4% wet of optimum in the borrow area, and the climate is wet, it is pointless requiring 98% density ratio at OWC∓1%. The specification would be more realistically 95% density ratio at optimum to optimum +4%, or specified as detailed in Section 14.6.3.
c) *Carrying out insufficient laboratory compaction tests*. The density ratio is obtained by comparing the density in place with the maximum dry density obtained in the laboratory. Ideally the soil for this laboratory compaction is sampled beside the density in place test. However, it is common practice to reduce the work involved by doing one laboratory compaction for every 2 to 4 or more density in place tests, assuming the soils are uniform. This is almost invariably not true and a small change in the maximum dry density can make the difference between acceptance and failure. Disputes often arise when such short cuts are used.
d) *Breakdown of materials during compaction*. Soils which contain pieces of weathered rock, or gravels which break down under compaction, will often give a higher laboratory maximum dry density if the compaction test is carried out on soil dug next to the density-in-place test hole, i.e. on soil already compacted and broken down by the roller, than if the test is carried out on material sampled direct from the borrow area. The result of this is that the density ratio calculated is lower when the recompacted soil maximum dry density is used, often resulting in rejection of the fill. Since the rollers are dealing with the soil from the borrow area, not recompacted soil, this is unreasonable. There are two solutions for this problem – either lower the density ratio standard and use the recompacted maximum dry density, or ensure that laboratory compactions are done on representative uncompacted material. The latter may be difficult because of material variability.
e) *Property change on drying*. Some soils change properties when dried in an oven or under lights. Halloysite clays are particularly prone to this, but most clays are

affected. It is desirable not to dry the soil used for the laboratory compaction test completely, but to the water content needed for testing.

f) *Vibration from nearby construction equipment* may affect the density obtained by sand replacement methods. This can be overcome by using the water balloon method or nuclear methods, or by testing when equipment is not operating nearby.

g) *Inadequate curing of samples*. Soils, particularly higher plasticity clays, need time for water added for laboratory compaction tests to evenly distribute throughout the sample. This is the reason why most standards require 12 hours to 24 hours 'curing' of the soil before compaction. In a construction situation this may be regarded as impracticable and not adhered to. As a result, compaction results may be inconsistent and subject to error.

h) *Specification of standard soil tests for 'gravelly' materials*. The standard tests can be corrected for the presence of gravel particles up to a reasonable limit. However, if the soil to be tested is largely gravel size, the potential errors are too great and larger size compaction moulds must be used, or a methods type specification adopted. Inexperienced persons may even specify a density ratio rather than density index (relative density) for granular soils. One needs only to observe the loosening effect of a compaction hammer on sand in a compaction mould to appreciate that this cannot work.

14.6.3 Compaction control – some other methods

Some other approaches have been developed for routine compaction control of clay soils, particularly for fine grained clay soils which are compacted wet of optimum water content. These include:

- *Specifying water content and a density ratio based on wet density*. The authors have used this approach for a dam constructed in Papua New Guinea, using halloysitic clay where previous experience had shown that it was very difficult to define maximum dry density and water content, i.e. the laboratory testing gave very variable results. The specification was written to require a water content between 44% and 50% (roughly OWC $\mp$3%) for the mean OWC and the required compaction dry density was 98% of the dry density achieved in the laboratory at the field water content. This wide range of water content was only practicable for the soil being used and would not normally be acceptable
- *Specifying undrained shear strength of the compacted earthfill*. Knight (1990) describes the use of this technique, which is common in dams constructed in the United Kingdom. Undrained strengths used range from $\approx$40 kPa to 110 kPa, with tests being carried out using triaxial tests on 100 mm diameter driven tube samples, or on remoulded samples. Lower bound and mean values were specified. Knight (1990) indicates that advantages of the method are that "measured strengths reflect design intention and testing is speedy". The authors' view is that the technique is useful where soils are to be compacted significantly wet of optimum, but that otherwise it is preferable to adopt a density ratio and water content specification. For most dams, the undrained shear strength of the core is not a critical issue, because stability is controlled largely by rockfill zones.

Chapter 15

Concrete face rockfill dams

15.1 GENERAL ARRANGEMENT AND REASONS FOR SELECTING THIS TYPE OF DAM

15.1.1 Historic development of concrete face rockfill dams

The development of concrete face rockfill dams (CFRD) has been described by Galloway (1939), Regan (1997), ICOLD (1989a), Cooke (1984, 1993, 1999, 2000), and Cruz *et al.* (2009). This is closely linked to developments in compaction of rockfill in dams as summarized in Table 15.1.

The first rockfill dam to have a concrete face was constructed in California in 1895. This followed on construction of timber faced dumped rockfill dams beginning in the 1850s.

These early CFRDs often had steep (0.54H:1V to 0.75H:1V) slopes, with a skin of hand placed rocks to stop the face from ravelling. Then dams of greater height, up to 100 m high (Salt Springs. Dam, California), were constructed with concrete faces and rockfill dumped in thick layers, often greater than 20 m or 35 m, and placed without compaction other than sluicing.

The design of faced rockfill dams was (and still is) mainly empirical, based on experience and judgment. The typical features of designs of CFRD up until the late 1950s are shown in Figure 15.1.

Many of these dams performed well. However several higher dams leaked excessively due to deformation of the concrete face, with resulting opening of joints and cracking. This could be attributed to the low modulus of the dumped rockfill, and to the detailing of joints, which allowed compression of horizontal and vertical joint fillers in the central part of the dam face (which is under compression) and resulted in increased opening of other joints, including the perimetric joint. The leakage did not endanger the dam stability but in some cases was unacceptably high for operating reasons.

Over the period 1955 to 1965 there was a general adoption of compaction of rockfill. This change was brought about by a realisation that dumping and sluicing of rockfill led to significant segregation, with the accumulation of larger rock at the base of the layer, leaving large voids and being particularly compressible. There was also a realisation that weaker rocks tended to lose strength on saturation leading to settlement if placed as dumped rockfill. Cooke (1984) and ICOLD (1989a) attribute the major change in approach to Terzaghi (1960a).

Table 15.1 Historical summary of the trend in design and rockfill compaction in CFRD and Earth and Rockfill dams (Hunter 2003, Hunter and Fell 2002, based on Galloway 1939, Cooke 1984, 1993).

Approximate Time Period	*Method of Placement and Characteristics of Rockfill*	*Comments*
Concrete Faced Rockfill Dams		
Mid to late 1800s to early 1900s	Dumped rockfill with timber facing	Early embankments constructed with timber facing. Typically of very steep slopes (up to 0.5 to 0.75H to 1V). First usage of concrete facing in the 1890s. Height limited to about 25 m.
1920s–1930s	Dumped in high lifts (up to 20 to 50 m) and sluiced, hand or derrick placed upstream rockfill zone. Sluicing relatively ineffective	Rockfill typically sound and not subject to disintegration. Dam heights reaching 80 to 100 m. For high dams cracking of facing slab and joint openings resulted in high leakage rates (2700 l/sec Dix River, 3600 l/sec Cogswell, 570 l/sec Salt Springs).
Late 1930s to 1960s	High pressure sluicing used. Rockfill still very coarse.	Cracking of face slab, particularly at the perimeter joint, and high leakage rates a significant issue with higher dams (3100 l/sec at Wishon, 1300 l/sec at Courtright).
From late 1960s	Rockfill placed in 1–2 m lifts, watered and compacted. Reduction in particle size. Usage of gravels and lower strength rock.	Significant reduction in post-construction deformations due to low compressibility of compacted rockfill. Significant reduction in leakage rates; maximum rates typically less than 50–100 l/sec. Continued improvement in plinth design and facing details to reduce cracking and leakage.
Earth and Rockfill Dams		
1900 to 1930	Dumped rockfill	Use of concrete cores with dumped rockfill shoulders at angle of repose. Limited use of earth core. Dam heights up to 50–70 m.
1930s to 1960s	Earth core (sloping and central) with dumped rockfill shoulders	Use of earth cores significant from the 1940s due to the difficulties with leakage of CFRD. Increasing dam heights up to 150 m.
From 1960s	Use of compacted rockfill. Typically placed in 1–2 m lifts, watered and compacted with rollers	Improvements in compaction techniques. Early dams compacted in relatively thick layers with small rollers. Gradual increase in roller size and reduction in layer thickness reduced the compressibility of the rockfill. Significant increase in dam heights in the mid to late 1970s, up to 250–300 m.

Terzaghi (1960b) also introduced the change from a deep cutoff trench in rock as shown in Figure 15.1, to the adoption of the use of a plinth, or toe slab, on the grounds that excavation for the trench could loosen and fracture the rock, making it more permeable, and that the plinth could be an adequate cutoff if founded on suitable rock, grouted and anchored to the rock with steel bars.

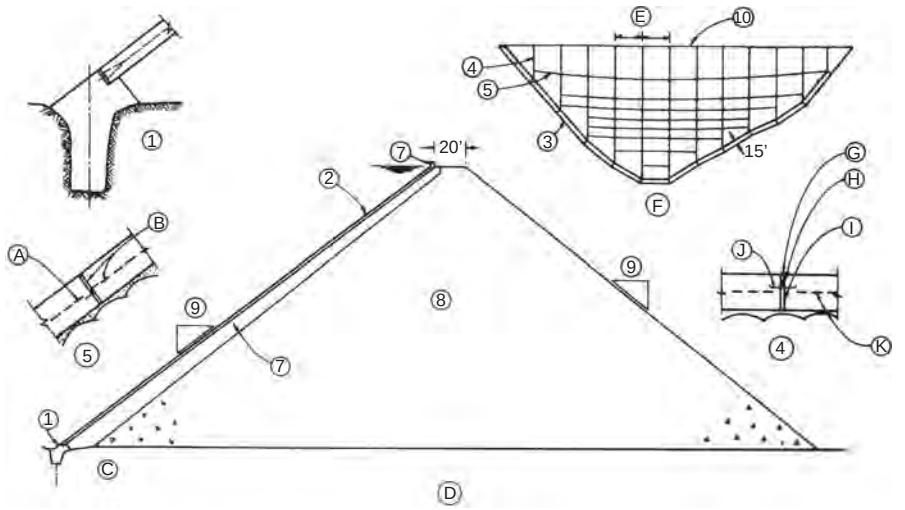


Figure 15.1 Features of early concrete face rockfill dam design (ICOLD, 1989a). (1) Cutoff trench, (2) Concrete face, (3) Plinth, (4) Vertical joint, (5) Horizontal joint, (6) Parapet, (7) Crane-placed large rock, (8) Dumped rockfill, (9) Slope, (10), Curved axis. (A) Reinforcement, (B) 1.9 cm redwood filter and Z water stop, (C) Grout curtain, (D) Cross section of dam, (E) 18 m (60 feet), (F) Elevation of face, (G) Mastic, (H) Pre-moulded asphalt, (I) Compressible joint filler, (J) U copper, (K) Reinforcement.

15.1.2 General arrangement – modern practice

Figure 15.2 shows the modern practice for zoning of CFRD constructed of sound, free draining rockfill, on a strong rock foundation.

The dam consists of:

Plinth. Reinforced concrete slab cast on sound, low permeability rock to join the face slab to the foundation.

Face slab. Reinforced concrete, preferably between 0.25 and 0.6 m thick, with vertical, some horizontal and perimetric joints to accommodate deformation which occurs during construction, and when the water load is applied.

Zone 2D. Transition rockfill, processed rockfill or alluvium, grading from silt to cobble size or, in more recent dams, from silt to coarse gravel size. The transition provides uniform support for the face slab and acts as semi-impervious layer to restrict flow through the dam in the event that cracking of the faceplate or opening of joints occurs.

Zone 2E. Fine rockfill, selected fine rock which acts as a filter transition between Zone 2D and Zone 3A in the event of leakage through the dam.

Zone 3A. Rockfill, quarry run, free draining rockfill placed in layers about 1 m thick. This zone provides the main support for the face slab and is compacted to a high modulus to limit settlement of the face slab.

Zone 3B. Coarse rockfill, quarry run, free draining rockfill placed in layers about 1.5 to 2.0 m thick. Larger rock may be pushed to the downstream face. This zone is

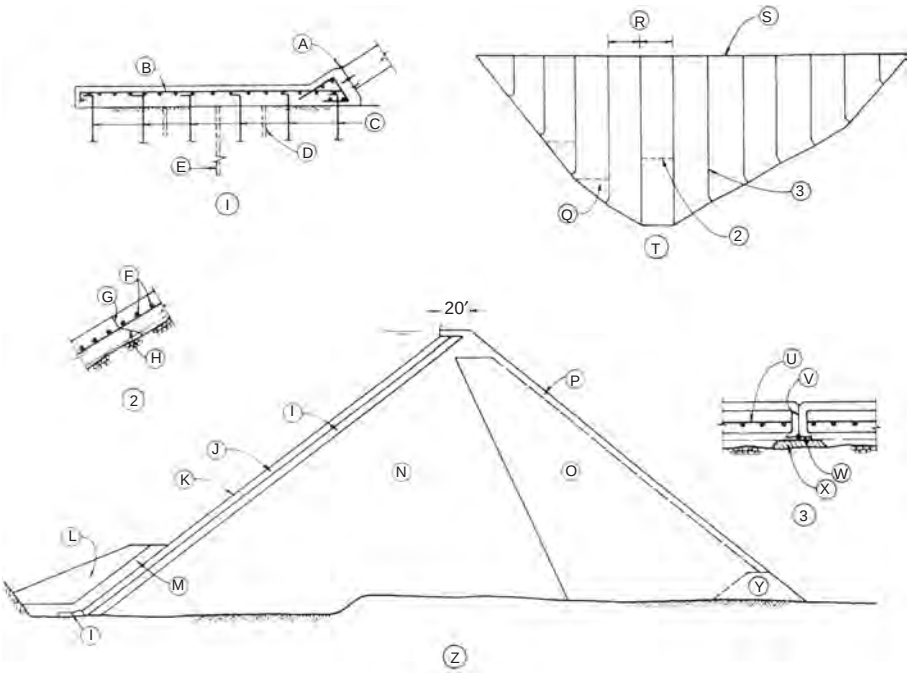


Figure 15.2 Modern practice for zoning of CFRD constructed of sound rockfill on a strong rock foundation (adapted from ICOLD, 1989a). (1) Plinth, (2) Horizontal joint, (3) Vertical joint. (A) Perimetric joint, (B) Steel reinforcement, (C) Anchor bars, (D) Consolidation grout holes, (E) Grout curtain, (F) Horizontal reinforcement, (G) Width form, (H) Broom joint, (I) Zone 2E, selected small rock placed in the same layer thickness as Zone 2D, (J) Zone 2D, processed small rock, (K) Concrete face, (L) Zone 1B, random, (M) Zone 1A, impervious soil, (N) Zone 3A, quarry run rockfill or gravel fill, about 1.0 m layers, (O) Zone 3B, quarry run rockfill or gravel fill, about 1.5 m to 2.0 m layers, (P) Available large size rock dozed to face, (Q) Starter slab, (R) 18 m, (S) Straight axis, (T) Elevation of face, (U) Horizontal reinforcement, (V) Surface painted with asphalt, (W) Copper water stop, (X) Mortar pad, (Y) Zone 3D, plus 0.3 m rockfill, (Z) Section of dam.

less affected by the water load than Zone 3B, so a lower modulus is acceptable. The thicker layers allow placement of larger rock.

The boundary between Zone 3A and 3B should be selected with care. There has been a trend to moving the boundary from what is shown in Figure 15.2 to the dam centreline and even upstream of the centreline, but Marulanda and Pinto (2000) warn that, at least for high dams, this may lead to larger settlements.

Zone 2F. Some modern dams include a Zone 2F filter zone beneath the perimetric joint as shown in Figure 15.3. This serves two main functions: (a) to act as a high modulus zone to limit deformation of the slab at the perimetric point; (b) to act to limit leakage flow in the event the joint opens. If a zone of earthfill (Zone 1A) is provided upstream of the slab, Zone 2F acts as a filter to that earthfill in the event the joint opens and leakage initiates.

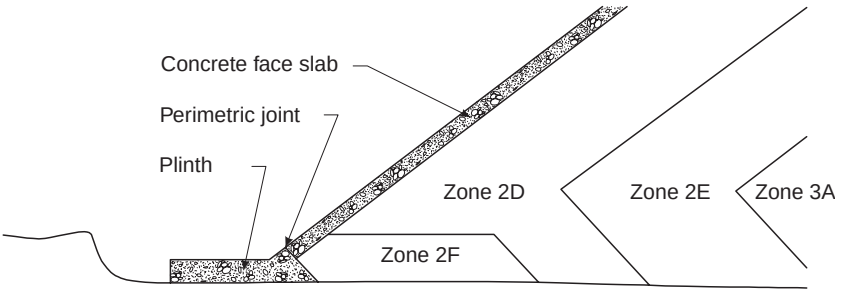


Figure 15.3 Detail of Plinth area showing Zone 2F.

This Zone 2F also provides protection against any potential for fines in any small defects in the foundation that might be lost due to the high hydraulic gradient across the line of the plinth. Zones 2F is usually a maximum size of 19 mm or 37 mm, with some silty fines, placed in thin (200 mm) layers. Suggested gradation limits for Zone 2F are given in ICOLD 2010. Zone 2F should be designed to be a no-erosion filter for and soil or weathered rock which may be eroded in the foundation beneath the plinth.

Many variations of this zoning are adopted to meet site conditions and the quality of construction materials available. As discussed in Section 15.5.1, non free draining rockfill may be used, provided free draining zones are incorporated to give any seepage water a controlled path to the dam toe. In some dams, low permeability earthfill is placed upstream of the face (Zones 1A and 1B in Figure 15.2) to control leakage. This is discussed in Section 15.5.5.

The zone designation discussed above is used throughout the book. The transition Zones 2D and 2E have been so designated to differentiate from Zones 2A, 2B and 2C used in earth and rockfill dams. There is no consistent terminology in use throughout the world.

Many dams include a concrete crest wall which reduces the quantity of rockfill required (see Section 15.3.4).

These developments, along with other refinements in design, have resulted in the earlier problems being overcome and acceptance of CFRD for the construction of many dams, including dams up to 233 m high. CFRD up to 300 m high are in the design phase.

However as described in Cruz *et al.* (2009) incidents have been experienced on several high dams due to damage to the face slab after reservoir filling. These included Tianshengqiao 1, Barra Grande, Campos Novos and Mohale dams. These incidents involved damage to the face slabs and subsequent leakage have been attributed to very large deformations of the face slab under water loading that has lead to high compressive stresses in the centre of the face slab with failure of the vertical compression joint and rupture of the face slab. In response to these incidents there have been refinements to the design of face slabs for high dams as described in Section 15.3.2.

15.1.3 Site suitability, and advantages of concrete face rockfill dams

CFRD are suited to dam sites with a rock foundation and a source of suitable rockfill. In many cases CFRD will be a lower cost alternative than an earth and rockfill dam. This is discussed in Sherard and Cooke (1987), and Fitzpatrick *et al.* (1985). Factors, which may lead to CFRD being the most economic alternative, include:

- The non availability of suitable earthfill.
- Climate. CFRD are suited to wet climates, which may give short periods in which earthfill can be placed. This can result in significant overall savings in schedule.
- Grouting for CFRD can be carried out independently of embankment construction, which may result in savings in overall time for construction.
- Total embankment fill quantities are likely to be smaller and side slopes steeper for CFRD than for earth and rockfill dams leading to reductions in the cost of fill and diversion tunnels.

Sherard and Cooke (1987) indicate that the cost of the concrete face is often less than the additional costs of earthfill and filters and more extensive foundation treatment for earth and rockfill dams. This has commonly been the case in Australia where a significant proportion of major embankment dams built after 1985 have been CFRD.

Sherard and Cooke (1987) point out that CFRD have generally been used for dams of moderate height (40 m) or higher. They suggest that CFRD may also be economic for lower dams if they have a long crest length because:

- The cost of foundation treatment for a long, low dam is high relative to the overall cost, but will be less for CFRD, because the CFRD requires a smaller width to be treated, than for earth and rockfill.
- The cost of filters in a long, low earth and rockfill dam is relatively high because the filters constitute a high percentage of the total volume.

They describe the use of CFRD for a 2000 m long 20 m high dam in Venezuela.

Varty *et al.* (1985) indicate that the Hydro-Electric Commission of Tasmania used CFRD for several smaller dams, ranging in height from 20 m to 44 m. They list faceplate construction techniques which are different from their normal practice for these smaller dams, including the elimination of starter bays, a lighter slip-form and a mobile crane on the dam crest.

These examples show that smaller dams can be economically constructed as CFRD, but probably where the economy of scale applies, such as a long crest, or a requirement to construct several dams, to offset the cost of setting up for slip-forming.

Sherard and Cooke (1987) and Cooke (2000) argued that CFRD should be considered for the very highest dams. Sherard and Cooke (1987) argue that CFRD have fundamental advantage over earth and rockfill dams in that there is no possibility of piping erosion of the earth core and over arch dams in that they rely on gravity for stability, not high strength abutments, and that the CFRD supports high abutments, rather than stressing them. They argue that there is no reason CFRD up to 300 m high could not be built and suggest that the jump in precedent from current heights (at that

time) is not excessive and is similar to practice in earth and rockfill dams. They suggest that conservative perimetric joint details would have to be used for very high CFRD, and wider, more conservatively designed, processed material used for Zones 2D and 2E to ensure homogeneity, lack of segregation and low permeability.

15.2 ROCKFILL ZONES AND THEIR PROPERTIES

15.2.1 Zone 2D – Transition rockfill

As the design of concrete face rockfill dams has developed and higher dams have been constructed, more emphasis has been placed on the grading and placement of the rockfill zone immediately below the concrete face slab. In CFRDs constructed in the 1960s the function of Zone 2D was seen as providing uniform support for the face slab. Specifications required that the Zone 2D was screened to remove all materials less than 25 or 50 mm. This was to ensure that, in the event of a leak in the face slab, there would be no fines to be washed away which might lead to loss of support of the face slab. However with this grading Zone 2D was very permeable and, rocks were easily dislodged during construction, and a rough, porous surface led to excess concrete being required for the face slab.

Beginning with the 110 m high Cethana Dam in 1971 (Wilkins *et al.*, 1973) there was a change in design approach which resulted in Zone 2D being specified as 'crusher run' or 'quarry run' (after passing through a grizzly) rockfill passing 150 to 225 mm (up to 300 mm). This resulted in a material with a lower permeability and a smoother and more stable surface on which to construct the face slab. Many dams have been successfully constructed in this way and this was the established practice up till about the mid 1980s.

Gradations for Cethana, Alto Anchicaya and Foz de Areia are shown in Table 15.2.

Later designs have specified a finer grading with Zone 2D not only meeting filter requirements with Zone 2E, but also having a specified requirement for a silty fines

Table 15.2 Zone 2D gradation specifications (Sherard 1985b).

	Cethana	*Alto Anchicaya*	*Foz do Areia*
Year	1971	1974	1980
Size % finer			
300 mm		100	
225 mm	100		
150 mm	80–100		100
100 mm			
75 mm	51–100	50–100	55–100
40 mm			
20 mm	15–63	25–60	18–65
5 mm	0–40	0–30	5–22
0.5 mm	0–17	0–5	0–8
0.075 mm	0–5		0–1

Table 15.3 Characteristics of Zone 2D in Brazilian CFRD (Sobrinho *et al.*, 2000).

Dam *Fill Type*	*Foz do Areia* *Crushed sound basalt*	*Segredo* *Crushed sound basalt*	*Itá* *Crushed sound basalt*	*Xingo* *Grizzlied sound and weathered granite/gneiss*	*Machadinho* *Crushed sound basalt*	*Itapebi* *Processed gneiss*
Geometry						
Bottom width (m)	13	8	10	12	10	12
Crest width (m)	4	5	3 + 4	4/6	3 + 4	3 + 4
Layer (thickness m)	0.4	0.4	0.4	0.4	0.4	0.4
Gradation						
Max. Size mm	100	75	75	100	75	100
25.4 mm % passing	50	45	60	70	50	80
4.76 mm % passing	12	20	25	44	15	45
0.150 mm % passing	1	2	5	10	7	11
0.075 mm % passing	0	0	1	7	2	7
Compaction						
Horizontal (passes/roller)	4/10-tonne vibratory	4/10-tonne vibratory	4/9 tonne vibratory	6/9 tonne vibratory	4/10 tonne vibratory	4/9 tonne vibratory
Upslope (passes/roller)	6 passes	4/static + 6/vibratory	extruded wall	4/static + 6/vibratory	extruded wall	extruded wall
Void Ratio	0.31	0.21	0.175		0.31	0.19
Density (t/m^3)	2.12	2.27	2.15	2.12	1.97	2.20
Performance During Construction	Adequate	Adequate	Adequate	Cracking + Settlement	Under Construction	Under Construction

content to reduce the permeability, and hence leakage, in the event of a face slab joint opening or the slab cracking.

Sherard (1985b) points out that quite large leakage had occurred at some dams constructed with crusher-run Zone 2D including 1800 litres/second at the Alto Anchicaya Dam on first filling. Sherard notes that, for such wide graded materials, with a small percentage of sand size material, segregation was a problem, and this could in part explain why such large leakage had occurred. He suggests that about 40% of sand size (passing 4.76 mm) is required to avoid the segregation. Sherard also shows that leakage through the dam resulting from a crack in the face slab, is controlled more by the permeability of Zone 2D, than by the crack aperture. Sherard concludes that Zone 2D should be a 4 m to 5 m wide zone.

The properties of the Zone 2D material placed during the construction of six major CFRD in Brazil (Sobrinho *et al.*, 2000) are summarized in Table 15.3. These dams have transition zones that range in width from 10 m at the bottom to 4 m at the crest.

Current practice follows the recommendations of Sherard (1985b) and reported in ICOLD (2010) and Amaya and Marulanda (2000) as shown in Table 15.4.

Table 15.4 Desirable specification for Zone 2D.

Size	*Sherard (1985b)* % Finer	*ICOLD (2010)* % Finer	*Amaya & Marulanda (2000)* % Finer
75 mm	90–100	90–100	90–100
37 mm	70–95	70–100	70–100
19 mm	55–80	55–80	65–100
4.76 mm	35–55	35–60	40–55
1.18 mm		18–40	
0.6 mm	8–30		
0.3 mm		6–18	
0.075 mm	2–12	0–7	4–8

Sherard (1985b) indicates that a grading for Zone 2D as shown in Table 15.4 will be 'stable', i.e. internally stable, not susceptible to washing out of the fines, and that a permeability of 10^{-6} m/sec would be achieved. Earlier Zone 2D are not internally stable, and will be susceptible to the finer particles washing from the zone, leading to a higher permeability and possibly some settlement of the face slab as the fines are eroded.

ICOLD (1989a) recommended a Zone 2D grading virtually the same as that suggested by Sherard. They indicated that a maximum of 10% to 12%, passing 0.075 mm is desirable, while giving 15% as the upper limit.

ICOLD (2010) modified these gradations to specify more completely the finer materials but in particular to reduce the % fines ICOLD (1989a) had recommended. This was to reduce the likelihood that the Zone 2D might hold a crack. They specify the fines passing 0.075 mm must be non-plastic. These are more in keeping with Sherard (1985b).

ICOLD (1989a) indicates that Zone 2D should be 4 m to 5 m wide, possibly wider for dams higher than 150 m. At Antamina Dam in Colombia (Amaya and Marulanda, 2000) where strict control of seepage is required, a Zone 2D width of 8 m has been adopted.

Sherard (1985b) acknowledges that the required Zone 2D grading may require processing and/or blending of materials, but points out that the incremental cost is small. Mori (1999) suggests the use of compacted alluvial gravel in Zone 2D to reduce deformation and cracking of the face slab.

Fitzpatrick *et al.* (1985) and Sherard (1985b) and Cooke (2000) point out that an added advantage of having Zone 2D graded as shown in Table 15.4, is that, if a crack forms in the face slab, it can be sealed by spreading silty sand over the crack and having the silty sand wash into it, with further erosion controlled by Zone 2D acting as a filter. This method has been used to reduce leakage on several dams including Shiroro Dam in Nigeria (Bodtman and Wyatt, 1985), Khao Laem Dam in Thailand (Watakeekul *et al.*, 1985) and Bailey (Cooke, 2000).

The authors strongly support the Sherard (1989b)/ICOLD (2010) approach to design of Zone 2D. Provided the gradation of the material is within the boundaries they propose and the materials are not gap graded (e.g. the gradation shown in Figure 15.4)

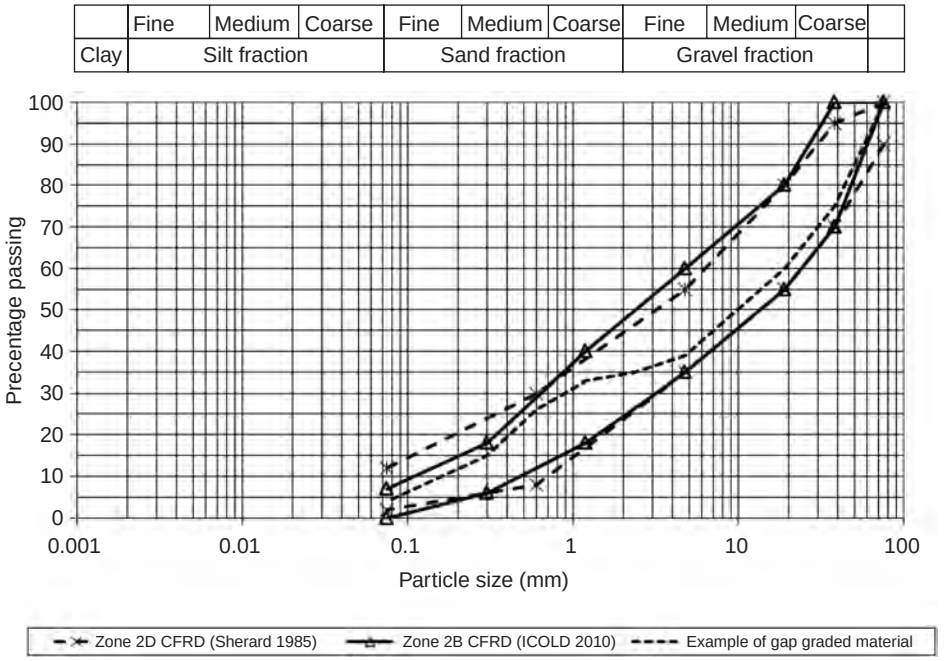


Figure 15.4 Sherard (1985b) and ICOLD (1989a) Zone 2D gradation limits. Example of gap-graded material also shown.

the materials will be internally stable using the Wan and Fell (2008) method. If the material is gap graded and internally unstable the finer fraction may erode leaving only the coarse fraction which may be too permeable to limit leakage to tolerable values.

It is critical to have a low permeability zone beneath the face slab because historically the incidence of unexpected leaks in CFRD is high. Zone 2D limits the leakage rate to what is tolerable for free draining rockfill to cope with without unravelling. The gradation of Zone 2D also facilitates intervention by spreading sand on the open crack as described above.

The finer side gradations are likely to be most effective and least subject to segregation during placement and to holding a crack. The % fines should however not exceed 7% because otherwise the material may hold a crack.

ICOLD (2010) (pg. 255–257) discusses problems with deep cracking of the Zone 2D material with 10% to 15% non-plastic fines during construction and operation of two CFRD's. During construction cracks were observed in the Zone 2D up to 60–100 mm wide and 3 m deep. This was caused by differential settlements of the rockfill zone. It would appear that in one of the dams (Xingo), the cracks in the Zone 2D were believed to have reopened during operation and this adversely affected its ability to limit flows through leaks in the face slab (a complete seal was not achieved when they dumped silty sand on the face). These seem to be important case histories and the

main reason why ICOLD modified their recommended curves to limit the fines to less than 7%.

When assessing the safety of existing dams with high fines content Zone 2D there should be consideration of the possibility of the Zone 2D holding a crack.

This zone is as critical to CFRD as filters are to earth core dams and any pressure to relax the requirements to reduce cost should be resisted. Contractors should be advised that it is processed material zone with a specific gradation and to price it accordingly.

Early 1960s' constructed dams which have the coarse grading Zone 2D have no flow limiting zone, and if the rockfill is relatively low permeability because of break-down on compaction (e.g. schist, phyllite, argillite, sandstone, siltstone) the rockfill may unravel or even become saturated leading potentially to instability.

15.2.2 Zones 2E, 3A and 3B – Fine rockfill, rockfill and coarse rockfill

15.2.2.1 *General requirements*

The basic requirements for rockfill in a CFRD are:

- The rockfill should be free draining to avoid build-up of pore pressure during construction, and to allow controlled drainage of water which might leak through the face slab.
- The rockfill should have a high enough modulus after compaction in the dam to limit face slab deflections under water load to acceptable values. Creep of the rockfill should also be small enough to avoid excessive longer term settlements.
- It should be readily available as a quarry run product with a minimum of wastage of oversize or undersize rock.

A wide range of rock types have produced satisfactory rockfill including granite, basalt, dolerite, quartzite, rhyolite, hornfels, limestone, gneiss, greywacke, andesite, welded tuff and diorite. Rocks such as sandstone, siltstone, argillite, schist and shale have been used but in some cases produce a non-free draining rockfill. This is discussed further in Section 15.5.1. Gravels have also been used with success and as discussed below, can lead to very high modulus fills. Cooke (1984) suggests that 'if blasted rockfill is strong enough to support construction trucks and the 10 tonne vibratory roller when wetted, it may be considered to be suitable for use in compacted rockfill'. He goes on to point out the need for drainage zones if the resulting rockfill is of low permeability.

Penman (1982) and Penman and Charles (1976) suggest that for rockfill to be considered 'free draining', it should have a permeability of at least 10^{-5} m/sec based on *in situ* tests in the rockfill. This is based on the requirement for adequate permeability to dissipate construction pore pressures in the rockfill, rather than having a high water discharge capacity from a leaking face slab. The authors would caution against thinking such low permeability rockfill is "free draining" as their experience is that rockfill with a permeability of 10^{-5} m/sec may not cope with the large potential leaks, e.g. resulting from face slab damage in earthquakes, and saturation of the rockfill may occur. This may lead to potential slope instability.

The authors are satisfied rockfill is free draining if water will flow rapidly from a test pit which penetrates the full layer thickness of the rockfill, when pumped in from a water tanker.

As discussed by Cooke (1984) and Sherard and Cooke (1987), rockfill placed in the normal way, i.e. dumped from a truck, and spread by a bulldozer, will result in segregation, with the coarser particles collecting at the base of the layer, and the finer rock and fines on the surface. Breakdown of the upper part of the layer during rolling creates even more stratification. Cooke (1984, 1993) and Cooke and Sherard (1987) point out that far from being a problem, this stratification is desirable because:

- Tyre wear on trucks is reduced and the smoother surface allows more rapid truck travel.
- The smooth surface facilitates the rolling operation, spreading the vibrating load and reducing roller maintenance compared to an irregular rocky surface
- The lower parts of layers have a high horizontal permeability, facilitating drainage of leakage or embankment overtopping during construction. The resulting average horizontal permeability is much higher than if the fines were distributed uniformly throughout a layer
- The layers create a variation in vertical permeability which will prevent buildup of pore pressures in the rockfill.

It should be noted that this stratification means that 'free draining rockfill' may allow water to pond on the surface of layers, provided the lower parts have higher permeability.

As pointed out by Cooke (1984), there is no need to scarify the surface of layers of compacted rockfill prior to placing the next layer.

The fourth author was shown in 1997 a video of a 30 m high rockfill (quarried) coffer dam in Brazil that had been built in 3–4 days, the crest just keeping ahead of a rising flood. At the start of the coffer dam construction, an impervious zone had been placed on the upstream face, but with the advent of a flood, the contractor decided to push ahead with the coffer dam as a simple rockfill dam. The video showed clearly that seepage appeared on the downstream side from the lower half of each layer. The rockfill placed in roughly 1 m layers and compacted only by construction traffic, did not start to ravel at any stage, except for the odd stone moved at abutment-dam contact lines. The behaviour of this coffer dam showed clearly how well an embankment of quality rockfill, 'compacted' in layers, will cope with extreme conditions without overtopping. In this case, the dam was deliberately breached before it overtopped.

A possible draw-back of this surface layer breakdown and hence the development of lower vertical permeability layers between layers is that for rockfill zones that are being relied upon to give vertical drainage capacity may not actually have that capacity. This condition is more likely to be related to cases where the CFRD does not have a deliberate low permeability zone directly beneath the face slab.

15.2.2.2 *Layer thickness and compaction*

Zones 3A and 3B are placed in layers of the order of 1 m and 1.5 m to 2 m thick respectively, which results in a gradation of permeability and a lower modulus for Zone 3B, which is acceptable since the water load is largely taken by Zone 3A. Rolling

for good quality rockfill is usually by 4 (up to 8) passes of a 10 tonne or more likely 12 to 15 tonne vibratory steel drum roller. Cooke (1993) indicates there is no evidence that rollers heavier than 10 tonne give better results. To avoid embankment deformation Mori (1999) notes that based on experience at Xingo (150 m high), Aquamilpa (187 m high) and Tianshengqiao 1 (178 m high) Zones 3A and 3B should not have a very different moduli of compressibility, should have a near vertical interface between them and a high embankment should not be built in stages. This is discussed in Section 15.6. Cooke (1993) suggests that lower strength (<30 MPa) rockfill should be compacted in thinner (0.6 m to 0.8 m) layers to encourage breakdown to give greater "strength" (a higher modulus). However the authors' experience is that this is likely to result in low permeability rockfill and zoning of the dam to have a separate free draining rockfill zone may be necessary.

Hydro Electric Commission of Tasmania (HEC) (Carter *et al.*, 2000) place Zone 3A in 1 m layers for good quality rockfill and 300 mm to 600 mm layers for lower quality but well graded material. Zone 3B is compacted in 1.5 m layers to take large rock.

Zone 2E is required to act as a filter to Zone 2D, and is therefore placed in relatively thin layers (0.4 m to 0.5 m, the same as Zone 2D). This also ensures a high modulus.

Zone 2E is obtained by selecting suitable material in the quarry, or by passing rockfill over a grizzly. The filter requirement (compared to Zones 2D and 3A) is usually readily achieved by this process, and further treatment is not required.

The rockfill grading in Zone 3A (and to a lesser extent in Zone 3B) is important. A well graded rockfill will compact to a higher modulus than a poorly graded fill, and the amount of fines needs to be limited if the rockfill is to be truly free draining.

Cooke (1984) suggests that the grading specification should be:

- Maximum size shall be that which can be incorporated in the layer and provides a relatively smooth surface for compaction.
- Not more than 50% shall pass 25 mm sieve.
- Not more than 6% shall be clay size particles (this is taken to mean silt and clay).

Cooke (1993) suggests not more than 10% finer than 0.075 mm).

In Sherard and Cooke (1987), it is suggested that it is better to specify:

- Not more than 20% finer than 4.76 mm
- Not more than 10% finer than 0.075 mm (i.e. silt and clay).

They suggest that if the rockfill has a higher proportion of fines, 'the final evaluation of suitability can be made on the trafficability of the rockfill surface when the material is thoroughly wetted. A stable construction surface under travel of heavy trucks demonstrates that the wheel loads are being carried by a rockfill skeleton. An unstable construction surface, with springing, rutting, and difficult truck travel, shows that the volume of soil like fines is sufficient to make the rockfill relatively impervious. Where the surface is unstable, the fines dominate the behaviour and the resulting embankment may not have the properties desired for a pervious rockfill zone'. As pointed out by Cooke (1993) the purpose of the specification is to be able to reject loads of predominantly soil or rock with large amounts of fines.

Sherard and Cooke (1987) point out that there is no technical need for rock in the rockfill to have a high compressive strength. In their experience, rockfill constructed of rocks with compressive strength of 30–40 MPa are no more compressible than those of higher strength. However as discussed in Section 15.2.3 the rock strength does have some influence on the rockfill modulus.

Rocks with very high compressive strength lead to higher quarrying cost and wear on equipment. They conclude that any rock with a (soaked) unconfined compressive strength of 30 MPa or more is adequate. Lower strength rocks may be used with special zoning provisions to provide sufficient drainage capacity as discussed in Section 15.5.1.

15.2.2.3 *Use of gravel as rockfill*

Cooke (1984) discusses the use of gravel (really sandy gravel or sandy gravel with some silt) as rockfill and maintains that, if available, this material can be very suitable. He points out that:

- Gravel is often more economically handled than rockfill (lower excavation and loading cost, and less wear on tyres and rollers).
- Compacted gravels commonly have high modulus of compressibility, up to 5 to 10 times that of some compacted rockfill. In his experience since face slab movements vary roughly inversely with the modulus, and directly with the square of height of the dam, gravel fills are desirable for higher dams.
- If significant fines content is allowed, then chimney drains, abutment drains, filters and intermediate drainage layers will be required.

Mori (1999) recommends that alluvial gravel should be used if possible in the upstream third of the dam to limit deformation and cracking of the face slab.

Amaya and Marulanda (1985) describe the use of gravel for the 125 m Golillas Dam in Colombia. In this dam the bulk of the fill (i.e. Zone 2 in Figure 15.5) was natural river gravel. The grading is shown in Table 15.5.

The Hydro-Electric Commission of Tasmania used gravels to construct the 82 m high Crotty Dam in 1991 (Figure 15.6). The Golillas Dam gravels (Figure 15.5) were made up of 70% sandstone, 20% siliceous shale and 10% siltstone and limestone. The gravels were washed for drainage zones in the dam. Sierra *et al.* (1985) describe the use of gravels in Salvajina Dam in Colombia. The 148 m high dam had its 'Zone 3A' constructed of gravels from gold mining dredging for Zone 2 as designated in Table 15.5.

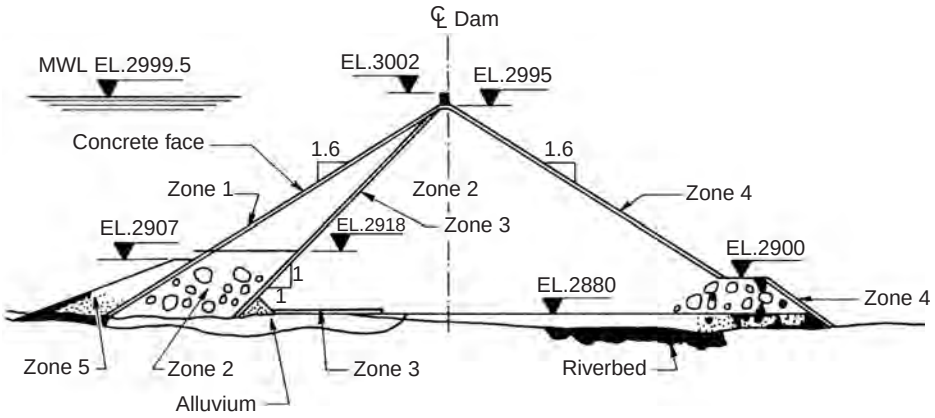
The range and average grading is given in Figure 15.4.

The gravels are usually placed in approximately 0.6 m thick layers and compacted with 4 passes of a 10 tonne steel drum vibratory roller, without water added. Water can make compaction difficult if the gravels are silty.

Very high moduli can be obtained as shown in Table 15.6.

As pointed out by Cooke (1984) gravel has been successfully used in several high earth and rockfill dams including Nurek (305 m), Oroville (244 m), Mica (244 m) and Bennett (183 m). Moduli of 365 MPa for Oroville and 551 to 689 MPa for Bennett were achieved despite use of smaller rollers than would now be adopted.

The Gouhou Dam in China (Chen, 1993), Chen and Zhang (2006), was constructed of sandy gravel with a finer grading envelope than the earlier dams



Maximum section

Zone	Description
1	Processed dirty gravels
2	Unprocessed dirty gravels
3	Processed clean gravels
4	Oversize from zone 2
5	Silty clays

Figure 15.5 Golillas Dam (Amaya and Marulanda, 1985).

Table 15.5 Grading of river gravels used as 'rockfill' in CFRD.

Dam	*Crotty*	*Golillas*	*Salvijina*	*Gouhou*
Year	1971	1978	1984	1992
Sieve Size	% Passing	% Passing	% Passing	% Passing
300 mm	100	100	100	100
150 mm	85–100 (98)[(b)]	60–100	65–100 (98)	100
75 mm	34–98 (85)	35–80	35–100 (80)	60–100 (90)
25 mm	26–70 (55)	20–50	10–80 (40)	55–90 (65)
4.76 mm	6–42 (16)	10–30	0–30 (8)	25–65 (42)
1.18 mm	2–22 (10)	4–22	0–25 (6)	15–45 (28)
0.075 mm	0–4 (2)	0–12[(a)]	0–12 (3)	5–10 (6)

Notes: (a) Average 8% for one source, 12% for second.
(b) Average in brackets.

(Table 15.5). The zoning of this dam is shown on Figure 15.8. There was little difference in grading between the zones II, III and IV. The dam failed by slope instability when leakage occurred through joints in the face, particularly at the join of the face slab and the wave wall which was below the reservoir normal FSL. The relatively fine fill, with no higher permeability zones as are built in to other gravel fill dams, contributed to the failure. Figure 15.9(a) and (b) show the failed dam and Figure 15.9(c) the crest wall detail.

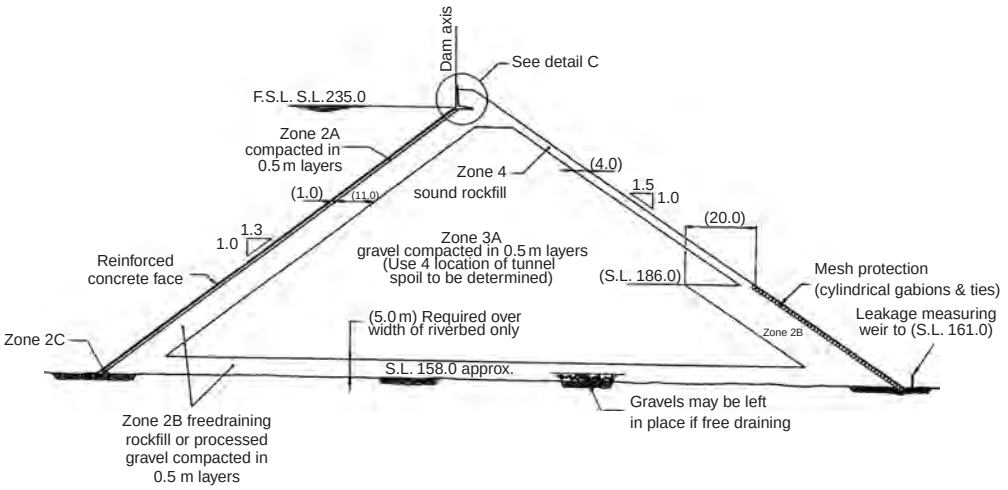


Figure 15.6 Crotty Dam (HEC, 1988).

15.2.3 Effect of rock properties, compaction and addition of water during compaction on modulus of rockfill

The modulus of compressibility (E) of the rockfill is dependent on the rock type, strength, shape and gradation of rock sizes in the rockfill and layer thickness. It is also dependent on the roller size and type, number of passes, whether water is added during compaction, the confining stresses on the rockfill and also the duration of loading, i.e. there is a creep component.

Table 15.6 summarizes the properties of compacted rockfill from a number of CFRDs. The rockfill moduli during construction (E_{rc}) and the pseudo modulus on first filling (E_{rf}) values have been calculated from observed settlements of rockfill during construction of the dam using cross-arm settlement gauges and from the observed deflection of the face slab on first filling. Unless otherwise stated, the values quoted are generally 'average', typical of rockfill at the lower half and centre of the dam. The moduli have been calculated with the simplified procedure shown in Figure 15.10, except that for E_{rc}, the stress distributions due to the shape of the embankment have been allowed for, using elastic solutions in Poulos and Davis (1974).

Fitzpatrick *et al.* (1985) calculate E_{rc} and E_{rf} from

$$E_{rc} = \gamma H d_1/\delta_s \tag{15.1}$$

$$E_{rf} = h\, d_2/\delta_n \tag{15.2}$$

where E_{rc} and E_{rf} are in MPa, γ = unit weight of the rockfill in kN/m^3, δ_s = settlement of layer of thickness d_1 due to the construction of the dam to a thickness H above that layer; δ_n = face slab deflection at depth h from the reservoir surface, and d is measured normal to the face slab as shown. δ_s, δ_n, H, d_1 and d_2 are all measured in metres.

Table 15.6 Summary of embankment and rockfill properties for CFRD (Hunter, 2003, Hunter and Fell, 2002).

	Embankment Dimensions		*Material Parameters/Properties of Rockfill*									*Rockfill Moduli*	
Dam Name	*Height, H (m)*	*L/H*	*Rockfill Source*1*	*Strength*2 Classification UCS (MPa)*	*C_u*	*d_{max} (mm)*	*% finer 19 mm*	*Dry Density (t/m^3)*	*Void Ratio, e*	*Layer Thickness (m)*	*Placement*	*E_{rc} (MPa), average*	*E_{rf} (MPa)*
Zone 3A Rockfill													
Aguamilpa	185.5	2.6	alluvium	(Very High)	85	600	34	2.22	0.18	0.6	4p 10t SDVR, in moist condition	305 (250 to 330)	770
Crotty	83	2.9	Gravels (Pleistocene)	(Very High)	70	200	48	2.54	0.20	0.6	8-12p 6-10t SDVR, watered	375 (113 to 636)	470
Golillas	125	0.9	gravels	(Very High)	125	350	40	2.135	0.24	0.6	4p 10t SDVR, water added	155 (145 to 165), arching likely	250
Salvajina	148	2.4	gravels	(Very High)	9.2	400	32	2.24	0.25	0.6	4p 10t SDVR, water added	205 (175 to 260) arching likely	500
Alto Anchicaya	140	1.9	hornfels	(Very High)	18	600	22	2.28	0.294	0.6	4p 10t SDVR, 20% water	138 (100 to 170), likely arching	375 (@ 30 to 40% height)
Bastyan	75	5.7	Rhyolite, SW to FR	(Very High)	42	600	25	2.20	0.23	1.0	8p (10t ?) SDVR, 20% water	130 (120 to 140)	290
Cethana	110	1.9	Quartzite	(Very High)	23	900	21	2.07	(0.27)	0.9	4p 10t SDVR, 15% water	160 (120 to 210), arching likely	300
Chengbing	74.6	4.4	Tuff lava	UCS = 80 Very High	10.4	1000	–	2.06	0.277	1.0	6p 10t SDVR, 25% water	43	110
Foz Do Areia	160	5.2	basalt (max 25% basaltic breccia)	UCS = 235 Very High	6	600	10	2.12	0.33	0.8	4p 10t SDVR, 25% water	47 (38 to 56)	80 (65 to 92)
Ita	125	7.0	basalt	(Very High)	11	700	12	2.179	0.308	0.8	6p 9t SDVR, 10% water	48	87 (83 to 91)
Kangaroo Creek	60	3.0	schist	UCS = 25 Medium to High	310	600	44	2.34	0.201	0.9 to 1.8	4p 10t SDVR, 100% water		140
Khao Laem	130	7.7	limestone	UCS < 190 Very High		900	–	–	–	1.0	4p 10t SDVR, 15% water lower in 70 m	59 (43 to 79)	130 to 240
Kotmale	90	6.2	charnockitic/ gneissic	(High to Very High)		700	–	2.20	–	1.0	4p 15t SDVR, 30% water	61 (47 to 87)	145 (135 to 155)
Little Para	53	4.2	dolomitic siltstone	UCS = 8 – 14 Medium	110	1000	35	2.15	0.223	1.0	4p 9t SDVR, no water lower half	21.5 (19.5 to 23.5)	–
Mackintosh	75	6.2	Greywacke, some slate	UCS = 45 High	52	1000	38	2.20	0.24	1.0	8p 10t SDVR, 10% water	45 (35 to 60)	63
Mangrove Creek	80	4.8	fresh siltstone & sandstone	UCS = 45 – 64 High	310	400	27	2.24	0.18	0.45 to 0.6	4p 10t SDVR, 7.5% water	55 to 60	–
Murchison	94	2.1	Rhyolite (SW to FR)	UCS = 148 Very High	19	600	22	2.27	0.234	1.0	8p 10t SDVR, 20% water	190 (170 to 205)	560 (485 to 640)

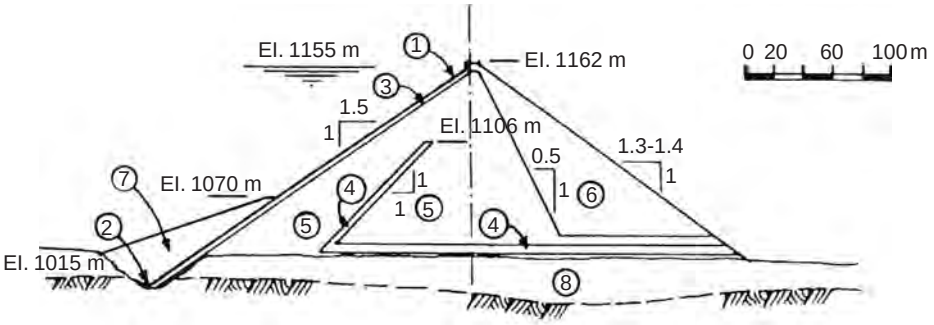
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Table 15.6 Continued.

	Embankment Dimensions		*Material Parameters / Properties of Rockfill*									*Rockfill Moduli*	
Dam Name	*Height, H (m)*	*L/H*	*Rockfill Source*1*	*Strength*2 Classification UCS (MPa)*	C_u	d_{max} *(mm)*	*% finer 19 mm*	*Dry Density (t/m^3)*	*Void Ratio, e*	*Layer Thickness (m)*	*Placement*	E_{rc} *(MPa), average*	E_{rf} *(MPa)*
Zone 3A Rockfill													
Reece	122	3.1	Dolerite	UCS = 80–370 Very High	10	1000	11	2.287	0.29	1.0	4p 10t SDVR, 5 to 10% water	86 (57 to 115)	190 (175 to 205)
Scotts Peak	43	24.8	argillite	UCS = 22 Medium to High	380	914	38	2.095	0.266	0.915	4-6p 10t SDVR, no water	20.5 (18.5 to 23.5)	59 (Zone 3A) 420 (Zone 2B)
Segredo	145	5.0	basalt (<5% basaltic breccia)	UCS = 235 Very High	7.4	-	-	2.13	0.37	0.8	6p 9t SDVR, 25% water	55 (arching likely)	175
Serpentine	38	3.5	Ripped quartz schist	(Medium to High)	210	152	69	2.10	0.262	0.6 to 0.9	4p 9t SDVR, not sure if water added	92 (46 to 142)	97 (94 to 100)
Shiroro	125	4.5	granite	(Very High)	32	500	22	2.226	0.20	1.0	6p 15t SDVR, 15% water	66 (61 to 71)	–
Tianshengqiao −1	178	6.6	Limestone, SW to FR	UCS = 70–90 Very High	15 to 20	800	–	2.19	0.23	0.8	6p 16t SDVR, 20% water	49 (40 to 57)	–
Tullabardine	25	8.6	Greywacke, some slate	UCS = 45 High	28	400	30.5	2.22	0.23	0.9 to 1.0	4p 10t SDVR, >10% water	74	170
White Spur	43	3.4	Tuff – SW to FR	(High to Very High ?)		1000	–	2.30	0.18 to 0.25	1.0	(4p 10t ?) SDVR, >10% water	180 (160 to 200)	340
Winneke	85	12.4	SW to FR	UCS = 66 High Siltstone	33	800	28	2.07	0.302	0.9	4-6p 10t SDVR, 15% water	55 (50 to 59)	104
Xibeikou	95	2.3	Limestone – FR	UCS = 240 Very High		600	–	2.18	0.284	0.8	8p 12t SDVR, 25 to 50% water	80 (60 to 100)	260
Xingo	140	6.1	granite gneiss	(High to Very High ?)	18	650	4 to 33	2.15	0.28	1.0	4p 10t SDVR, 15% water	34 (30 to 39)	76 (73 to 80)
Zone 3B Rockfill													
Mangrove Creek (Zone 3B)	80	4.8	weathered to fresh siltstone & sandstone	UCS = 26–64 High	330	450	32	2.06 t/m^3	0.26	0.45	4p 10t SDVR, dry of OMC	46 (36 to 56)	–
Salvajina (Zone 3B)	148	2.4	weak sandstone and siltstone	(Medium ?)	45	600	32	2.26	0.21	0.9	6p 10t SDVR, water added	62 (likely arching)	
Tianshengqiao −1 (Zone 3B)	178	6.6	Mudstone	UCS = 16–20 Medium	40	600	20 to 35	2.23	0.21	0.8	6p 16t SDVR, 20% water	37 (32 to 42)	–
Foz Do Areia (Zone 3B)	160	5.2	mix basalt & basaltic breccia	UCS = 235 High to Very High	14.2	–	–	1.98	0.27	0.8 for ID 1.6 for IC	4p 10t SDVR, 25% water	32 (29 to 38)	–
Aguamilpa (Zone 3B)	185.5	2.6	ignimbrite	UCS = 180 Very High	22	700	–	–	–	1.2	4p 10t SDVR	36 (25 to 45)	–
Ita (Zone 3B)	125	7.0	Breccia and Basalt	(High to Very High)	13.3	750	15	2.066	(0.33 to 0.39)	1.6	4p 9t SDVR, no water	24 (14 to 46)	–
Khao Laem (Zone 3B)	130	7.7	limestone	UCS < 190 Very High		1500	–	–	–	2.0	4p 10t SDVR, 15% water lower in 70 m	30	–

Segredo (Zone 3B)	145	5.0	basalt (<5% basaltic breccia)	UCS = 235 High to Very High	10.2	–	–	2.01 t/m3	0.43	1D −0.8 1C −1.6	4p 9t SDVR, no water	28 (25 to 33), likely arching	–
Xingo (Zone 3B)	140	6.1	Sound and weathered granite gneiss	(Medium to Very High ?)	80	750	15 to 60	2.1	0.31	2	6p 10t SDVR, no water	13 (12 to 14)	–
Dumped Rockfill													
Courtright	97	2.8	Granite	(Very High)	(<7)	1750	–	(1.8)	0.47	8 to 52	dumped and well sluiced	–	–
El-Infiernillo (Zone 3B)	148	2.3	diorite & silicified conglomerate	UCS = 125 Very High	13	600	22	1.85	0.47	0.6 to 1.0	4p D8 dozer, no water	39 (27 to 48)	–
Lower Bear No. 1	75	3.9	granodiorite	UCS = 100 −140 Very High	8 to 10	2000 to 3000	low	–	–	max 65	dumped and well sluiced		21
Lower Bear No. 2	46	5.7	granodiorite	UCS = 100 −140 Very High	8 to 10	2000 to 3000	low	–	–	max 36.5	dumped and well sluiced		40
Wishon	90	11.3	granite	(Very High)	(<7)	1500 to 2000	–	1.80	0.47	8 to 52 (variable)	dumped and well sluiced	–	–
Dix River	84	3.7	Limestone & Shale (?)	(Very High)	low	–	–	–	–	21	dumped and poorly sluiced	–	–
Salt Springs	100	4.0	granite	UCS = 100 −130 Very High	low (<10)	2000 to 3000	low	(1.88)	0.41	5 to 52	dumped and poorly sluiced		20
Cogswell	85.3	2.1	Granitic Gneiss	UCS = 45 High	7	1300	5	2.05	0.37	7.6 (46 m max)	dumped dry (no water)		44
El-Infiernillo (Zone 3C)	148	2.3	diorite & silicified conglomerate	UCS = 125 Very High	<13	>600	–	1.76	0.54	2.0 to 2.5	Dumped and spread, no water	22 (17 to 27)	–

Legend:
H = dam height
L = crest length
C_u = uniformity coefficient (d60/d10)
d_{max} = average maximum particle size
E_{rf} = deformation modulus during first filling
E_{rc} = secant modulus during construction (average)
4p 10t SDVR = 4 passes of 10 tonne smooth drum vibrating roller
*1 FR = fresh, SW = slightly weathered
*2 rock strength classification to AS 1726-1993
(…) in strength classification, Cu, dry density and void ratio columns indicate estimation
% water = % by volume
– indicates unknown



Maximum section

Zoning	Maximum size in - (cm)	Thickness layer in - (cm)
① Face slab	–	–
② Toe slab	–	–
③ Zone 1 – compacted semipervious	4 (10)	18 (45)
④ Zone 2A – drain	16 (41)	24 (60)
⑤ Zone 2 – compacted gravels	12 (30)	24 (60)
⑥ Zone 4 – compacted rockfill	24 (60)	36 (90)
⑦ Zone 5 – impervious	1-12(2.5.30)	12 (30)
⑧ Alluvium	–	–

Figure 15.7 Salvajina Dam (Sierra *et al.*, 1985).

Equation 15.1 does not allow for the distribution of vertical stress in the dam due to the shape of the dam. E_{rf} is therefore not a true modulus of the rockfill, and should only be used as discussed below to estimate the face slab deformation.

The following points can be made:

- The use of high strength rock does not guarantee a high modulus, e.g. Foz do Areia Dam. The relatively low modulus for this dam is due to poor grading, i.e. a lack of sand size particles (Pinto *et al.*, 1985), and is typical of basaltic rocks in that region. The void ratio of the compacted rockfill is a guide to the rockfill behaviour (note that Foz do Areia Dam has a relatively high void ratio of 0.33. Most rockfill have a void ratio less than 0.25).
- Low strength rocks, which break down significantly, e.g. Kangaroo Creek, Little Para and Mangrove Creek, can give quite high moduli, but it must be remembered that they will probably also give low permeability.
- The highest moduli are achieved for gravels, where the rounded shape limits crushing of the contact points between particles of the fill. Conversely the high stresses on the contact points in the poorly graded basaltic fill help cause the low modulus.
- Well graded high strength rockfill can also give high moduli, e.g. Murchison, Bastyan and Reece.

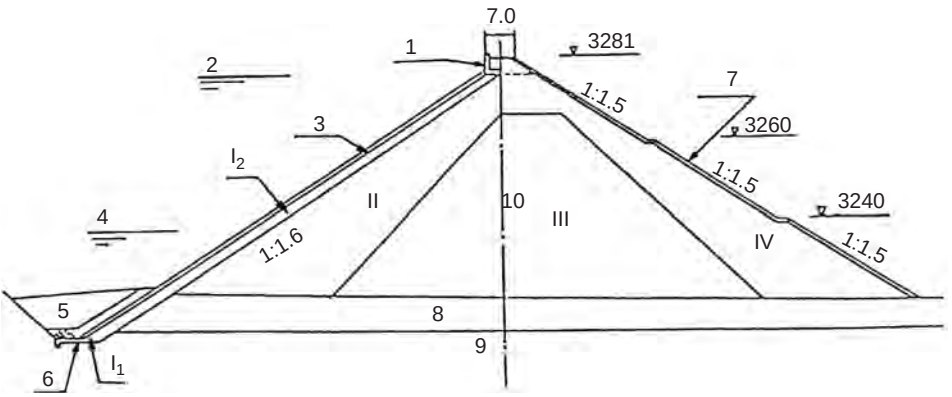
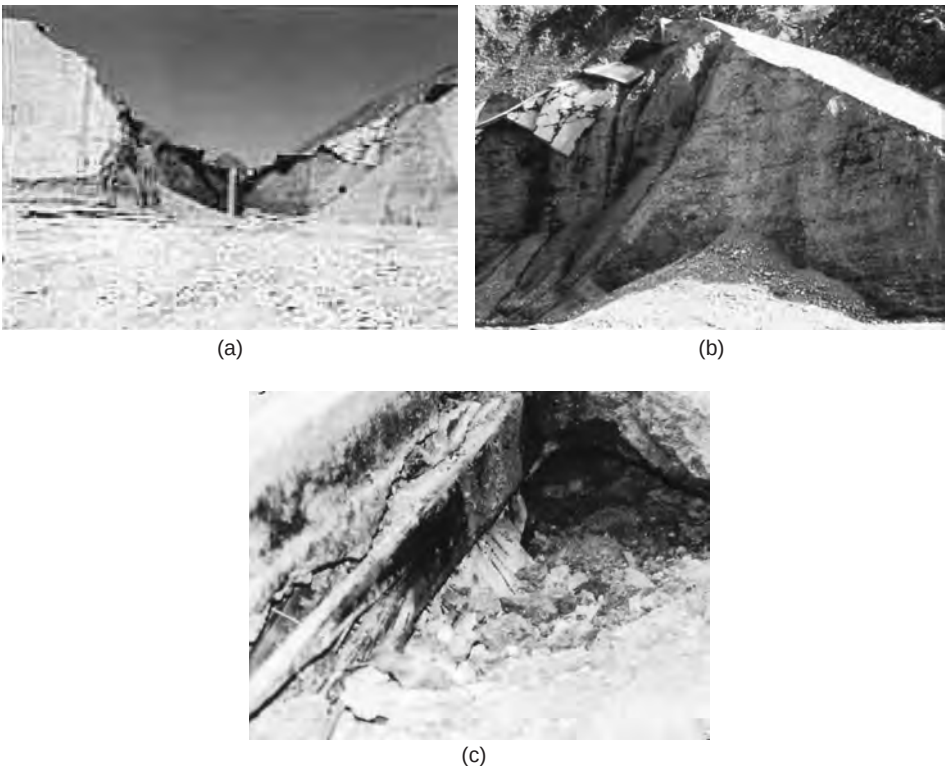


Figure 15.8 Gouhou Dam (Chen, 1993). 1. Crest wall; 2. Normal water level; 3. Face slab; 4. Dead water level; 5. Random fill; 6. Clay; 7. Masonry; 8. Alluvium; 9. Dam axis.



(a) (b)

(c)

Figure 15.9 Gouhou Dam failure (a) view from downstream, (b) section, (c) crest wall details. Photographs courtesy of Professor Chen Zuyu.

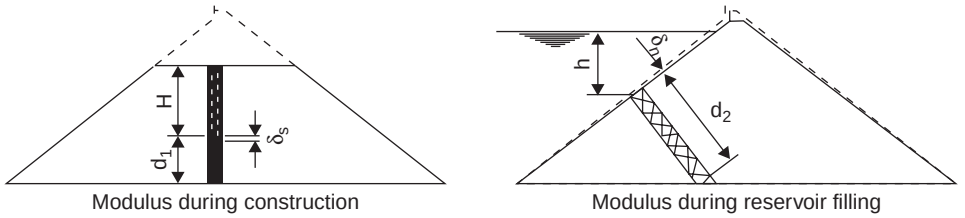


Figure 15.10 Simplified method of calculating rockfill modulus during construction, and pseudo modulus during reservoir filling (Fitzpatrick *et al.*, 1985).

- Additional compaction (e.g. Murchison 8 passes) and thin layers (e.g. Alto Anchicaya) can lead to higher moduli.
- The modulus is dependent on layer thickness, e.g. at Foz do Areia and Khao Laem dams where Zone 3B, compacted in layers twice the thickness of Zone 3A had moduli 60% to 80% of that for Zone 3A.
- The pseudo modulus on first filling is commonly 2 to 3 times that observed during construction. This is important as the face slab displacement is inversely proportional to the 'first filling' modulus and is discussed further in Section 15.2.5. Hunter (2003), and Hunter and Fell (2002) studied this and concluded that the apparent increase in modulus is an artefact of the method of calculation, rather than real. They showed that the layering effects within each rockfill layer had little effect and that, while the mean stresses increase from the construction condition as the water load is applied, the change in deviator stress is small.

It will be noted that in all dams water was added to the fill, usually at a rate of 15% to 25% of the rockfill volume. This results in reduced compressibility, although the improvement may be marginal in some cases.

Cooke and Sherard (1987) conclude that: *'(1) For most hard rocks and CFRD of low to moderate height, the addition of water has negligible influence on the dam behaviour. (2) For high dams and for rocks which have significantly lower unconfined compression strengths when tested in saturated conditions (than when tested dry), water should probably be added routinely for the upstream shell (Zone 3A). (3) For rocks with questionably high contents of earth and sand size particles, water should nearly always be used. For dirty rock, the water softens the fines so that the larger rocks can be forced into contact with each other by the vibrating roller.'* Cooke (1993) relates the need for adding water to the water absorption capacity of the rock, with little benefit being achieved for low (<2%) water absorption.

As pointed out by Sherard and Cooke (1987), it is not intended that the application of water will wash fines into the voids and hence the use of a high pressure nozzle is not necessary. Adding water in the truck prior to dumping on the surface is practicable and economical (Varty *et al.*, 1985; Cruz *et al.*, 2009).

Some indication of likely rockfill behaviour will be obtained from consideration of rock type, unconfined compressive strength and jointing in the rock in the quarry.

The best guide to rockfill behaviour in the investigation phase of a project will be to construct test fills, and possibly carrying out large diameter plate bearing

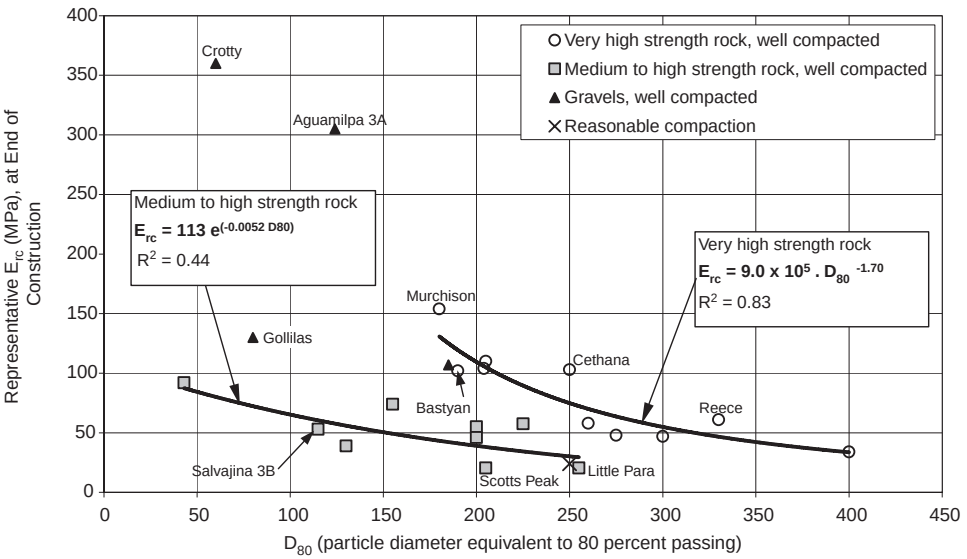


Figure 15.11 Representative secant modulus of compacted rockfill at the end of construction E_{rc} versus rockfill particle size and rock unconfined compressive strength (Hunter 2003, Hunter and Fell, 2002, 2003c).

tests on the compacted fill. This coupled with measurement of grading and void ratio of the resulting rockfill and consideration of the height of the dam, shape of the valley and comparison with other projects, should enable a reasonable guide to expected rockfill modulus. The method for estimating the modulus is described in Section 15.2.4.

15.2.4 Estimation of the modulus of rockfill

Hunter (2003), Hunter and Fell (2002, 2003c) developed a method for estimating the secant modulus of rockfill during construction (E_{rc}) and pseudo modulus on first filling (E_{rf}) based on analysis of monitoring data, mostly for CFRD.

15.2.4.1 *Estimation of the secant modulus E_{rc}*

Hunter and Fell (2002, 2003c) recommended that the following steps be followed:

(a) Determine the representative secant modulus at the end of construction E_{rc} from the D_{80} size (size for which 80% is finer) and unconfined compressive strength of the rock in the rockfill using Figure 15.11.

The representative secant modulus at the end of construction, E_{rc}, is for Zone 3A type rockfill, i.e. placed in layers 0.9 m to 1.2 m thick, compacted with 4 to 6

passes of a 10 tonne smooth drum vibratory roller and water added, and applies to the stresses:

1400 kPa for the very high strength, well-compacted rockfill
800 kPa for the medium to high strength, well-compacted rockfill
1500 kPa for the well-compacted gravels.

The stresses are calculated as simple vertical overburden stresses.

The D_{80} size should be obtained from construction records, rolling trials or estimates based on particle size from samples from test pitting into the existing rockfill. Unconfined compressive strength (UCS) ranges are (AS1726, 1993): very high 60–200 MPa; high 20–60 MPa; medium 6–20 MPa; Low 2–6 MPa.

(b) For Zone 3B or other thicker layer rockfill; apply a correction factor of up to 0.5 (for rockfill placed in layers up to 2 m thick) to obtain a representative E_{rc}. This correction factor is based on the ratio of E_{rc} (of Zone 3A to Zone 3B) from six field cases. There is not sufficient data to be prescriptive regarding this correction factor. As a guide, a correction factor of 0.5 would apply to rockfill placed in 2 m layers without the addition of water and compacted with 4 to 6 passes of a 10 tonne vibratory roller (i.e. reasonably compacted rockfill). A correction factor of 0.75 would apply for rockfill placed in 1.5 m to 1.6 m layers with the addition of water and compacted with 4 to 6 passes of a 10 tonne vibratory roller (i.e. reasonably to well compacted rockfill).

(c) To account for the non linearity of the stress-strain relationship for rockfill, estimates of modulus for stress levels less than or greater than the representative E_{rc} are done by:

- For very high strength rockfill apply a linear correction of ±7.5% per 200 kPa to the E_{rc} estimated from Figure 15.11 for a vertical stress of 1400 kPa. Apply positive corrections for decreasing stresses and negative corrections for increasing stresses. The applicable range is 400 kPa to 1600 kPa.
- For medium to high strength rockfill apply a linear correction of ±6% per 200 kPa to the E_{rc} estimated from Figure 15.11 for a vertical stress of 800 kPa (applicable range is 200 kPa to 1200 kPa).

(d) For medium strength rockfill apply a multiplication factor of 0.7 to the E_{rc} value determined from the equation for medium to high strength rockfill.

(e) Tangent moduli can be estimated from the secant moduli after correction for the stress levels.

The following points should be noted:

- C_u, the uniformity coefficient for the particle size distribution curve, is implicitly allowed for in the D_{80} value. Generally, decreasing C_u is observed for increasing D_{80}.
- For materials compacted with larger rollers (e.g. 13–15 tonne deadweight vibrating rollers) the data does not indicate an increase in moduli for the greater compactive effort. This may be due to greater material breakdown under the heavier rollers and a resultant reduction of D_{80}.
- For weathered rockfill the intact strength will be lower than for fresh rock, which will result in a decrease in secant moduli, but this will be countered by the greater

breakdown in particle size, which will give an increase in moduli associated with a reduction in D_{80}.

- If testing on the proposed rockfill material indicates a significant reduction in UCS on wetting is likely and only limited water has been, or is proposed to be, used in construction, the rockfill is likely to be more susceptible to settlements on wetting due to rainfall and flooding. There is insufficient data to advise on how to quantify this settlement.
- Only a limited number of cases of well-compacted gravels were available; insufficient to undertake analysis. These generally have significantly greater secant moduli at any given stress level in comparison to quarried, very high strength rockfill. Where the gravel is of finer size (e.g. Crotty Zone 3A) the use of a large scale laboratory testing that virtually encompasses the field particle size distribution would provide suitable estimates of moduli, but only if the laying and density in the field are reflected in the laboratory testing. Alternatively plate load tests on gravels compacted in field trials could be used.
- Monitoring should be undertaken during the embankment construction as a check on the pre-construction estimation of deformation.

Data to allow calculation of the moduli of dumped rockfill during construction or first filling was not available to Hunter (2003). It could be anticipated that the modulus would be much lower than for compacted rockfill. Dumped rockfill has a much greater susceptibility to collapse on saturation (e.g. Cogswell Dam (Bauman, 1958)) settled 5.4% following a heavy rainstorm which saturated the rockfill which had been dumped without sluicing). Back-analysis of the deformation of a narrow central core earth and rockfill dam built of dumped, sluiced, somewhat weathered basalt rockfill implied an equivalent secant modulus of 5 MPa for the rockfill on the upstream side, but probably higher on the downstream side. Rockfill at El Infiernillo dam which was dry dumped and spread in 2 m lifts gave a modulus of 17–27 MPa (average 22 MPa). At Ita dam, rockfill dumped into 10 m of water gave an estimated modulus of around 15 MPa to 19 MPa.

For assessing the effects of embankment raising it is recommended that moduli for the rockfill be determined from the deformation performance during construction, if that is available. Alternatively, tangent modulus versus applied vertical stress can be obtained from integration of this stress-strain curve. The variation can then be modelled by considering the embankment in a series of layers of varying moduli. For a more rigorous analysis numerical modelling can be used.

15.2.4.2 *Estimation of the first filling 'pseudo modulus' E_{rf}*

Hunter (2003), Hunter and Fell (2002, 2003c) showed that there was a relationship between the ratio of the pseudo modulus on first filling (E_{rf}) to the representative modulus during construction (E_{rc}) with embankment height and upstream slope as shown in Figure 15.12.

It should be noted that E_{rf} is a pseudo modulus, only to be used in estimates of face slab deformation using the simplified method shown in Figure 15.10 but allowing for the stress distribution using the Poulos and Davis (1974) method. For finite element analyses the modulus should be obtained as detailed in Section 15.2.4.1.

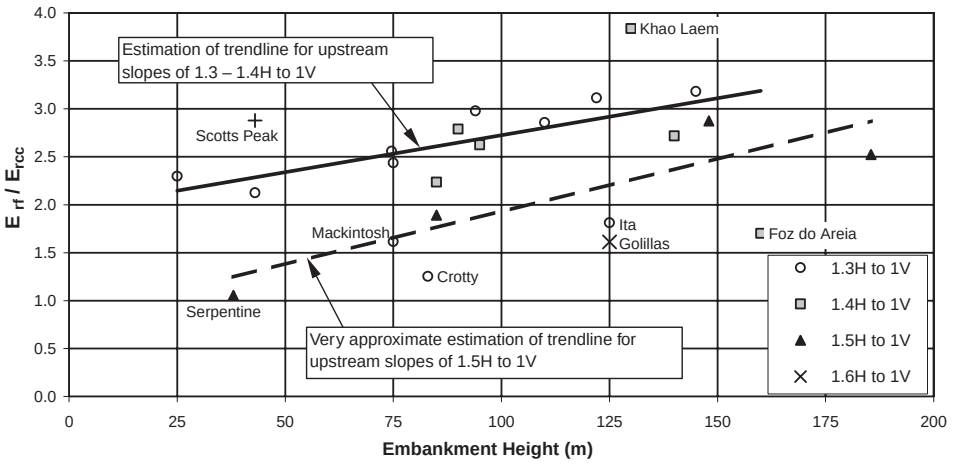


Figure 15.12 Ratio of pseudo modulus on first filling and construction rockfill modulus (E_{rf}/E_{rc}) versus embankment height (Hunter 2003, Hunter and Fell, 2002, 2003c).

The reasons for a number of the outliers on Figure 15.12 are detailed in Hunter and Fell (2002, 2003c). They mostly relate to complications arising from complex zoning (Ita, Crotty, Scott's Peak) and localised narrow section of the valley causing arching (Khao Laem). The apparently higher modulus on first filling implied by high E_{rf}/E_{rcc} values is due to the actual stress paths in the rockfill giving an initial reduction in deviator stress, as filling begins, and an overall relatively small increase in deviator stress on filling, as well as the simplifying assumptions made in the analysis.

Numerical analyses were carried out to assess the hypothesis put forward by Cooke (1984) that the layer of rockfill would have a higher horizontal modulus than vertical due to the high degree of compaction in the upper part of each layer, but these analyses showed such was not the case.

The steps to estimate E_{rf} are:

(a) Estimate the ratio E_{rf}/E_{rcc} based on the embankment height and upstream slope angle using Figure 15.12. Trend lines are given for upstream slopes of 1.3–1.4H to 1V, and 1.5H to 1V, which cover most CFRD designs.
E_{rcc} is the E_{rc} value estimated from Figure 15.11, adjusted for vertical stress such that it is representative of the average vertical stress in the lower 50% of the rockfill in the central region of the embankment.

(b) Estimate E_{rc}, the representative secant moduli at end of construction, of the Zone 3A rockfill from the methods outlined above. The E_{rc} value should be adjusted for vertical stress such that it is representative of the lower 50% of the rockfill in the central region of the embankment.

(c) The E_{rcc} values used in the derivation of Figure 15.12 were not corrected for arching effects due to valley shape because valley shape is potentially likely to affect both E_{rc} and E_{rf}, and would therefore be taken into consideration in the E_{rf}/E_{rcc} ratio.

Table 15.7 Approximate stress reduction factors to account for valley shape (Hunter 2003, Hunter and Fell 2002).

		Stress Reduction Factor (embankment location)			
W_r/H Ratio (river width to height)	*Average Abutment Slope Angle (degrees)*	*Base (0 to 20%)*	*Mid to Low (20 to 40%)*	*Mid (40 to 65%)*	*Upper (65% to crest)*
0.2	10 to 20	0.93	0.95	0.97	1
	20 to 30	0.88	0.92	0.96	0.98
	30 to 40	0.82	0.88	0.94	0.97
	40 to 50	0.74	0.83	0.91	0.96
	50 to 60	0.66	0.76	0.86	0.94
	60 to 70	0.57	0.69	0.82	0.92
0.5	<25	1	1	1	1
	25 to 40	0.93	0.95	0.97	1
	40 to 50	0.91	0.92	0.95	0.05–1.0
	50 to 60	0.87	0.88	0.93	0.05–1.0
	60 to 70	0.83	0.85	0.9	0.05–1.0
1	All slopes	0.95–1.0	0.95–1.0	1	1

Hence E_{rc} estimates derived from Figure 15.11 must be adjusted by dividing by the stress correction factors in Table 15.7 to give E_{rcc}.

(d) E_{rf} can then be estimated by multiplication of the value with the E_{rf}/E_{rcc} ratio.

The method applies to compacted rockfill. There is insufficient data on gravel fill dams to draw a trend line, but it appears that E_{rf}/E_{rcc} may be smaller for gravel fill than rockfill.

As will be apparent from the scatter of data in Figure 15.12, this method is approximate. It is most likely to be in error where the dam has complex zoning, with varying moduli in the zones, and in situations where valley shape effects are significant. The trend line for the upstream slope of 1.5H: 1V is particularly uncertain, being based on a limited number of cases.

15.2.4.3 *Effect of valley shape*

Several authors including Pinto and Filho Marques (1998), and Giudici *et al.* (2000) have concluded that the valley shape can have a significant effect on the settlements, because of 3 Dimensional effects shedding vertical stresses to the valley sides. This has been assessed empirically, aided (for Giudici *et al.*, 2000) with 3D numerical analyses.

Hunter (2003) and Hunter and Fell (2002) assessed this along with the other factors, and used simple 2D finite difference models to assess the likely effects of valley shape on calculated moduli. This showed that the effects were probably not as great as assumed by others, with the other factors such as particle size masking the 3D effects. If critical, the effects should be modelled using 2D and preferably 3D numerical analyses, but ensuring the modelling allows for the dam to be "built" in layers, since to "build" it numerically in one stage leads to an over-estimation of the 3D effects. For approximate analysis the stresses in the dam can be calculated using Table 15.7.

15.2.5 Selection of side slopes and analysis of slope stability

When the CFRD is constructed of hard, free draining rockfill, the upstream and downstream slopes are fixed at 1.3H to 1V or 1.4H to 1V, which corresponds roughly to the angle of repose of loose dumped rockfill. With slopes within the above range, ravelling of the faces should be prevented.

When gravel is used for the dam 'rockfill' zones, flatter slopes are needed to prevent ravelling of the face. Usually 1.5H:1V has been adopted in these cases although 1.6H:1V has been used.

When low permeability rockfill has been used flatter slopes have sometimes been adopted, e.g. 1.5H:1V for Mangrove Creek Dam (MacKenzie and McDonald, 1985). If foundation strengths dictate, flatter slopes may also be required, e.g. 2.2H to 1V was used for Winneke Dam (Casinader and Watt, 1985). Cooke (1999) confirms these values.

Haul roads may be needed on downstream slopes, or defined berms may be incorporated in the face. In these cases steeper slopes between the 'berm' located by the haul road may be used, e.g. 1.25H:1V was used for Foz do Areia Dam (Pinto *et al.*, 1985).

In some cases, the upper 10 m to 15 m of the dam may be steepened to as much as 1.25H to 1V to provide the camber of the crest (Fitzpatrick *et al.*, 1985).

As pointed out by Sherard and Cooke (1987) and Fitzpatrick *et al.* (1985), the stability of the slopes in the dam is not usually analysed. This is in recognition of the fact that well designed and constructed CFRDs have no pore pressures in the rockfill and will remain stable under static loads when constructed to the slopes described above. However, some existing dams with non-free draining rockfill may experience some degree of saturation of the rockfill if the face slab is damaged, e.g. in an earthquake, especially if there is no low permeability zone beneath the face to control seepage into the embankment. For these estimates must be made of potential pore pressures in the embankment allowing for the zoning, and nature of the rockfill including anisotropy of permeability and analyses of stability carried out.

If stability analysis is to be carried out, knowledge of the shear strength properties is required. The determination of rockfill properties is discussed in Section 6.2.

The deformations of CFRD under earthquake loading is discussed in Chapter 12.

15.3 CONCRETE FACE

15.3.1 Plinth

The principal purpose of the plinth (or 'toe-slab') is to provide a 'watertight' connection between the face slab and the dam foundation. Figure 15.13 shows a number of types of plinth designed to accommodate particular conditions.

Plinth width

The plinth is usually founded on strong, non erodible rock which is groutable, and which has been carefully excavated and cleaned up with a water jet to facilitate a low permeability cutoff. For these conditions the plinth width is of the order of 1/20 to

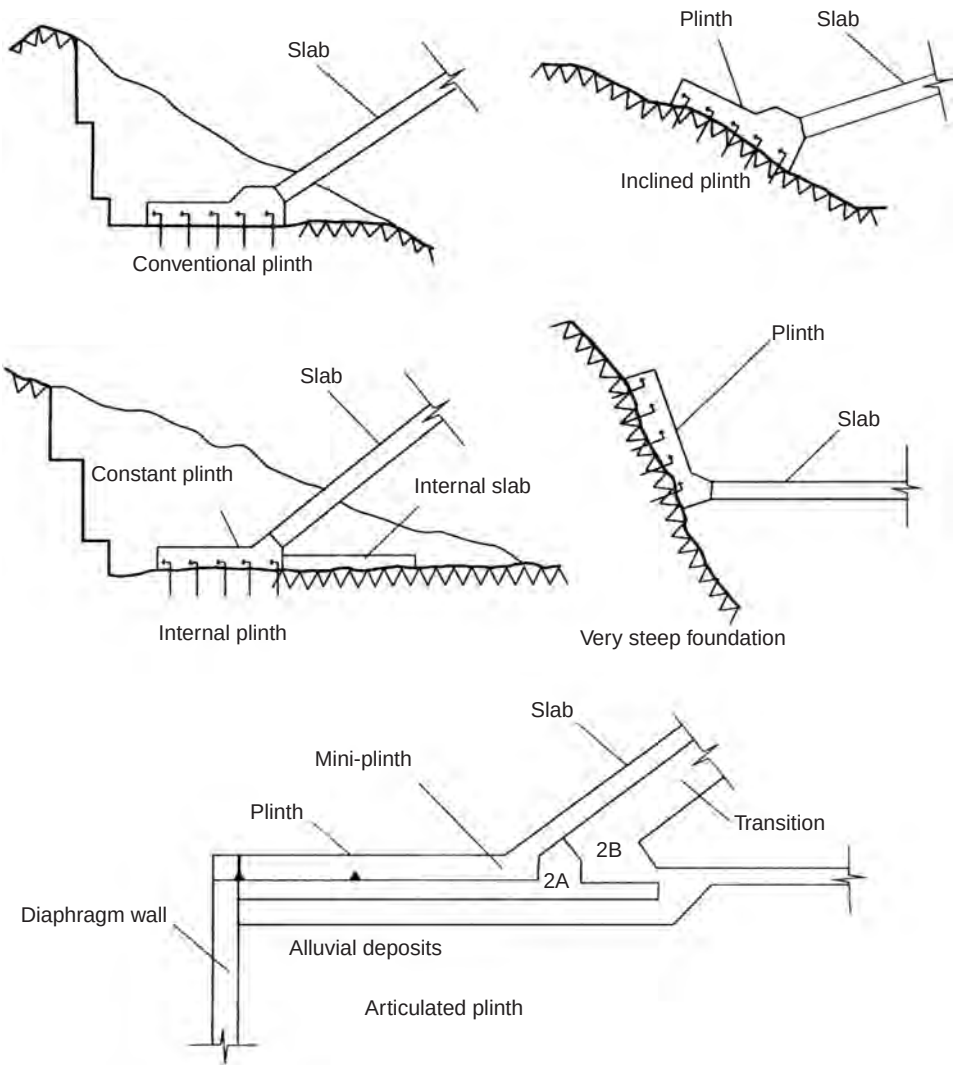


Figure 15.13 Types of plinth (Cruz *et al.*, 2009).

1/25 of the water depth (ICOLD, 1989b; Cooke and Sherard, 1987). Marulanda and Pinto (2000) suggest 1/10 to 1/20 depending on rock conditions.

Cruz *et al.* (2009) suggest that the plinth width may be designed to give acceptable gradients which are related to Rock Mass Rating (RMR) and indicate the criteria have been used for a number of dams. It is not clear how these criteria have been developed other than by precedent. The authors are not convinced that erodibility can be related to RMR and it is suggested that that method is treated with caution.

ICOLD (2010) include the method shown in Table 15.8 which seems to have parameters which are more relevant to erosion. It is suggested this be taken as a guide,

Table 15.8 Acceptable seepage gradient beneath the plinth related to foundation conditions (ICOLD, 2010, from Sierra, 1989).

A	*B*	*C*	*D*	*E*	*F*	*G*	*H*
I	Non erodible	1/18	>70	I to II	1 to 2	<1	1
II	Slightly erodible	1/12	50 to 70	II to III	2 to 3	1 to 2	2
III	Erodible	1/6	30 to 50	III to IV	3 to 5	2 to 4	3
IV	Highly erodible	1/3	0 to 30	IV to V	5 to 6	>4	4

where
A = Foundation type
B = Foundation class
C = Minimum ratio: Plinth width/Depth of water, full reservoir
D = Rock Quality Designation RQD, in %
E = Weathering degree: I equals sound rock, VI equals residual soil
F = Consistency degree: 1 equals very hard rock; 6 equals friable rock.
G = Weathering Macro Discontinuities per 10m.
H = Excavation classes:
1 = requires blasting
2 = requires heavy rippers; some blasting
3 = can be excavated with light rippers
4 = can be excavated with dozer blade

but the authors would prefer to also consider the width in relation to the potential for erosion of the rock beneath the plinth taking account of the orientation and persistence of open defects or features such as clay seams or other erodible material, the erosion properties of the features, and then take measures as detailed in Section 15.5.2 to control the erosion.

Up the dam abutment, the width is changed according to the water head. This is done in several steps (not gradually) for construction convenience. The minimum width has generally been 3 m, although Cooke and Sherard (1987) suggest that for dams less than 40 m high on very good rock, 2 m could be used.

Cooke (2000), Marulanda and Pinto (2000) and Cruz *et al.* (2009) indicate that a recent evolution in design is to reduce the length of the plinth to say 3 m to 5 m, and maintain an overall seepage gradient beneath the plinth by using an un-dowelled reinforced slab under the rockfill as shown in Figure 15.13 (labelled "internal slab"). This can give economies in rock excavation.

Plinth thickness

The minimum plinth thickness is usually between 0.3 m and 0.4 m, and 0.6 m for medium height dams, but Cruz *et al.* (2009) indicate it may be may be up to 1.00 m for the lower plinths of high dams (>120 m). The actual thickness is usually more because of the need to fill over-excavation, and to make up for irregularities in the topography. Where this extra concrete is significant it is common to construct the plinth in two stages; the first stage is to fill the irregularities.

Figure 15.14 and Figure 15.15 show plinth designs for Mangrove Creek, Boondooma, Cethana and Reece dams. These designs are still typical for CFRDs.

Plinth stability

It is necessary to ensure that the plinth is stable under the imposed forces. For a plinth of normal thickness there is adequate friction resistance on the base, unless there are unfavourably oriented low strength bedding planes, joints or shears in the foundations. The plinth is usually anchored to the rock with grouted dowels, which are generally 25 mm to 35 mm diameter, reinforcing steel bars, 3 m to 5 m long and are installed at 1.0 m to 1.5 m spacing. Cooke (2000) indicates 25 mm bars at 2 m spacing are adequate, that the anchor design is usually empirical, and that the bars are grouted full length into the rock and hooked on to the layer of reinforcing steel in the plinth. The anchors must have at least 25–30 mm of thick structural grout cover in their drilled holes and be centralised in those holes. The anchors are provided nominally to prevent uplift during grouting, although Cooke and Sherard (1987) claim uplift will not develop in most cases.

For plinths which are thicker than normal, due to over-break or irregularities in the foundation, the stability of the plinths should be analysed assuming the uplift pressure under the slab is zero at the downstream toe and varies linearly to full reservoir head at the upstream toe. No support should be assumed from the face slab on the understanding that the perimetric joint may have opened, and no support from the rockfill should be allowed for, since significant displacement into the rockfill would be necessary to mobilize the resistance (Marulanda and Pinto, 2000).

The plinth must be stable against sliding and overturning. A proper assessment of the sliding friction angle should be made depending on the rock type, orientation of weak defect surfaces etc., not just a check against some arbitrary 'sliding factor' or assumed friction coefficient.

It may be necessary to install additional buttress concrete downstream to maintain stability.

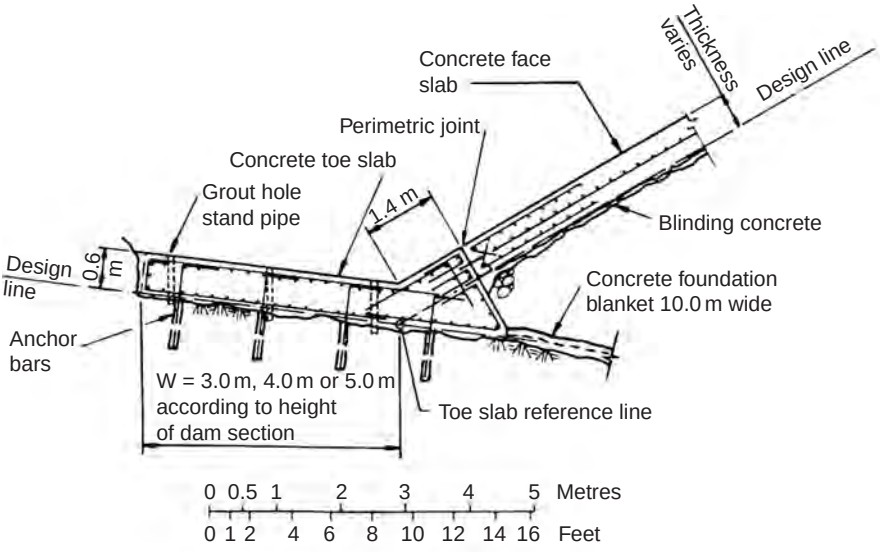
Detailing of zoning around plinth

If the plinth is high, a possible further problem is that excessive settlement of the face slab may occur and disrupt the perimetric joint. Problems were encountered in Golillas Dam due to such movements (in this case the valley sides were near vertical), (Amaya and Marulanda, 1985). Particular attention should always be paid to compacting the rockfill near the plinth and finer rockfill compacted in thinner layers may be adopted (Fitzpatrick *et al.*, 1985). However, one must also be careful not to obtain a modulus significantly higher than the rockfill or differential settlement may still occur. Some use a Zone 2F as shown in Figure 15.3, or Zone 2A as shown in Figure 15.36 in this area. These finer materials are more readily compacted to give a high modulus.

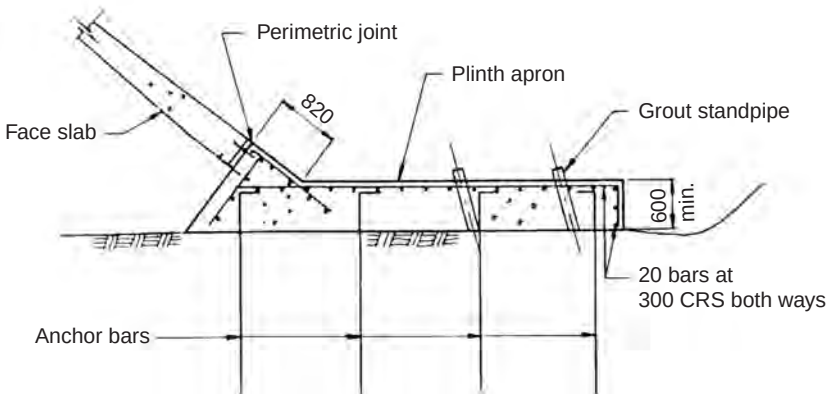
To ensure that the face slab displaces normal to its plane, and is not subject to bending, the Hydro-Electric Commission (Fitzpatrick *et al.*, 1985) required a minimum of 0.9 m of rockfill under the face slab, see Figure 15.15.

Plinth set out

The plinth is laid out as a series of straight lines selected to suit the foundation and topography. The angle points are not related to the vertical joints in the slab, and will


Thickness varies
Design line
Concrete face slab
Perimetric joint
Concrete toe slab
Grout hole stand pipe
1.4 m
Blinding concrete
Design line
0.6 m
Concrete foundation blanket 10.0 m wide
Anchor bars
W = 3.0 m, 4.0 m or 5.0 m according to height of dam section
Toe slab reference line
0 0.5 1 2 3 4 5 Metres
0 1 2 4 6 8 10 12 14 16 Feet

(a)



(b)

Figure 15.14 Plinth designs for a) Mangrove Creek Dam, b) Boondooma Dam (Mackenzie and McDonald, 1985, Rogers, 1985).

be a compromise between added excavation and face slab thickness and simplicity of construction.

The plinth may be laid out to be horizontal in a section normal to the centreline of the plinth (as for Cethana Dam, Figure 15.15(a)), or section normal to the plinth line (as in Reece, Figure 15.15(b)). The latter gives a smaller volume of excavation for the

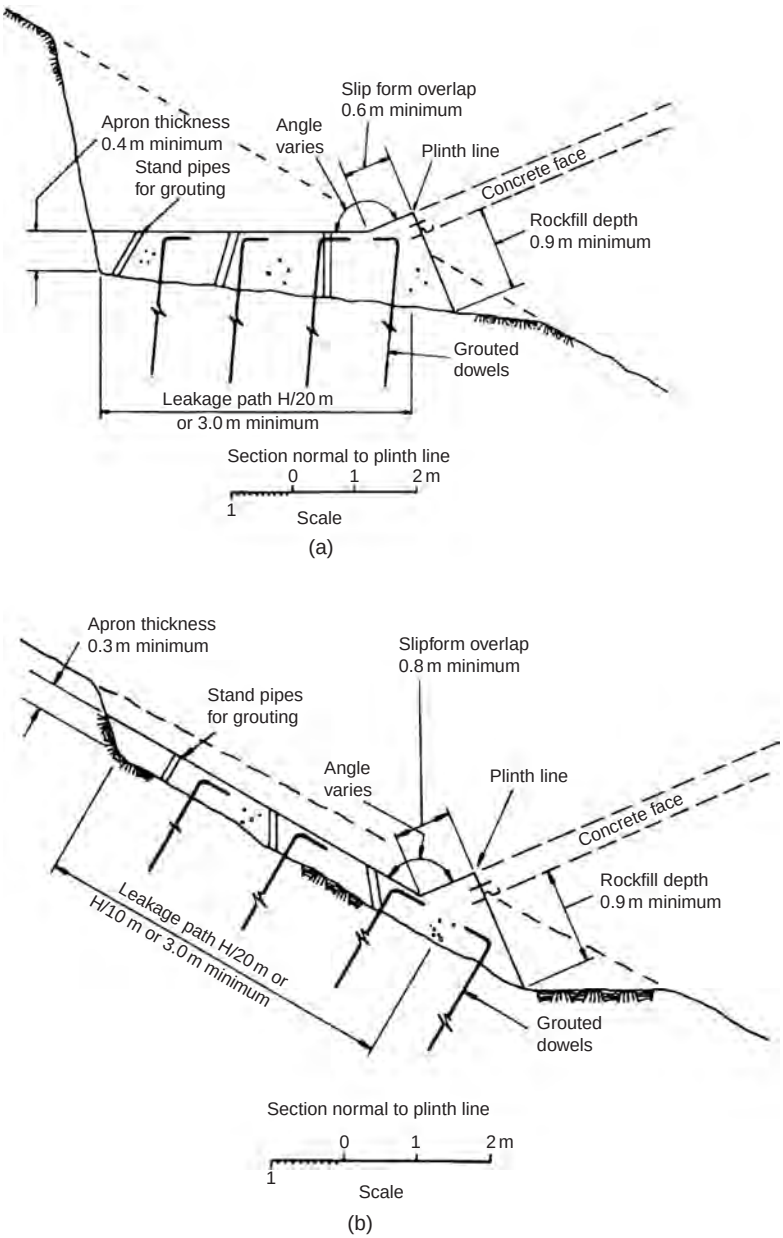


Figure 15.15 Plinth designs for (a) Cethana Dam, (b) Reece Dam (Fitzpatrick *et al.*, 1985).

plinth, but concreting and grouting costs were increased due to more difficult forming and access for drill rigs. Varty *et al.* (1985) indicate that the HEC reverted to the Cethana type layout for the 80 m high Crotty Dam. This layout was also adopted for

the Foz do Areia Dam because, although excavation was increased, this was compensated by better control of rockfill under the perimetric joint, a more straightforward concreting scheme and simpler drilling of grout holes (Pinto *et al.*, 1985). Cooke (2000) gives some suggestions on plinth layout.

Plinth reinforcement

The plinth is reinforced to control cracking due to temperature and to spread out and minimise cracks which may tend to develop from any bending strains from grouting. As well, for crack control it will also be important to minimise the Portland cement content in the concrete, keeping in mind all the time that required design strength must still be met.

ICOLD (1989a) and Cooke and Sherard (1987) indicate that a single layer of steel, 100 mm to 150 mm clear of the upper surface, with 0.3% steel each way is adequate, although the authors would favour a somewhat higher steel ratio closer to 0.6% if only one layer of steel is to be used. Earlier designs had a lower layer of steel but the advantages of the added stiffness are outweighed by the difficulty of cleaning the rock prior to placing the concrete. Cooke & Sherard (1987) and Cruz *et al.* (2009) indicate that longitudinal reinforcing steel should be carried through construction joints, rather than using formed joints with water stops.

15.3.2 Face slab

15.3.2.1 *Face slab thickness*

The face slab thickness is determined from past experience. ICOLD (1989a), Cooke and Sherard (1987) and Cooke (2000) recommend that:

- For dams of low and moderate height (up to 100 m): Use constant thickness = 0.25 m or 0.30 m.
- For high and/or very important dams: Use thickness = 0.3 m + 0.002H where H = water head in metres. These are minimum thicknesses; average thicknesses will be greater due to irregularities in the compacted Zone 2D. They assume a well constructed Zone 2D as detailed above. Earlier dams constructed on compacted rockfill were based on 0.3 m + 0.002H to 0.3 m + 0.004H, but improved construction methods have allowed a reduction to the above recommended values.

HEC used a face thickness of 250 mm up to a dam height of 75 m, 300 mm for higher dams plus local thickening near perimetric joint (Carter *et al.*, 2000). Cruz *et al.* (2009) indicate that for the design of two CFRD dams about 200 m high the thickness = 0.30 + 0.002H for the upper 100 m of the dam, and = 0.005H for where the dam was >100 m high. This is based upon requiring a constant hydraulic gradient < 200. They link this to analysis of seepage through face slab cracks as discussed in Casinader and Rome (1988). Just as with the plinth concrete, it will be important to control the cracking in the concrete, so the design of the concrete mix should aim to minimise the Portland cement content, consistent with the need to meet the design compressive strength. The use of flyash replacement would seem to have a lot of benefits, given that concrete with a flyash content has proved to be more impermeable than

with just Portland cement. As well, the flyash helps to reduce the peak temperatures at placement and usually improves workability for lower water/cement ratios.

15.3.2.2 *Reinforcement*

Steel reinforcement is provided to control cracking due to temperature and shrinkage. In general the face slab is under compression.

ICOLD (1989a) and Cooke and Sherard (1987) recommend the use of 0.4% reinforcing steel in each direction, with possible reduction to 0.3% or 0.35% in areas of the slab which will definitely be in compression, while retaining 0.4% within about 15 m of the perimeter. Cooke (1997, 1999, 2000), recommends the use of 0.3% horizontal, 0.4% vertical and 0.4% both ways within 15 m of the perimetric joint. However he points out that several experiences of cracks within about 20 m of the perimeter and near changes in the plinth slope, from relatively steep to flat upper slope, may justify 0.5% each way in these areas. Cooke (2000) recommends 0.5% in areas of major change in plinth slope.

The reinforcing steel is placed as a single mat at or just above the centreline. The area of steel is calculated on the theoretical minimum thickness (Cooke and Sherard, 1987 and ICOLD, 1989a).

The reinforcing has generally been structural grade reinforcing steel (e.g. Australian Standard S230), but more recently high yield grade deformed bar (e.g. Australian Standard 410Y) has been used and is desirable if available. The percentage of steel is the same regardless of steel type as the object is to control strain.

Reinforcement is provided in the face and plinth at perimetric joints, and in some cases across vertical joints which are likely to be under compression, to control spalling. Examples are shown in Figure 15.14a, Figure 15.16 and Figure 15.17.

15.3.2.3 *Vertical and horizontal joints*

Horizontal joints

ICOLD (1989a) indicates that design practice at that time did not include horizontal joints, except construction joints in which the reinforcing steel is carried through the joint without water stops. This is still current practice (Cruz *et al.*, 2009). Details of such joints are shown in Figure 15.19 and Figure 15.21. This was adopted because, when horizontal joints with water stops were used, it was difficult to obtain good quality concrete around the water stops and some joints experienced spalling under compression and mild rotation.

The Hydro-Electric Commission retained a horizontal contraction joint for Reece Dam, to reduce thermal shrinkage and face cracking.

Vertical joints

Vertical joints are generally provided at 12, 15, 16 or 18 m spacing depending on construction factors. For smaller dams, narrower spacing is desirable, e.g. 6 m.

Most CFRDs have been constructed with each of the vertical joints being a construction joint with a copper water stop at the base as shown in Figure 15.16 and Figure 15.18 and 15.21(a). The joints are generally painted with asphalt, not filled

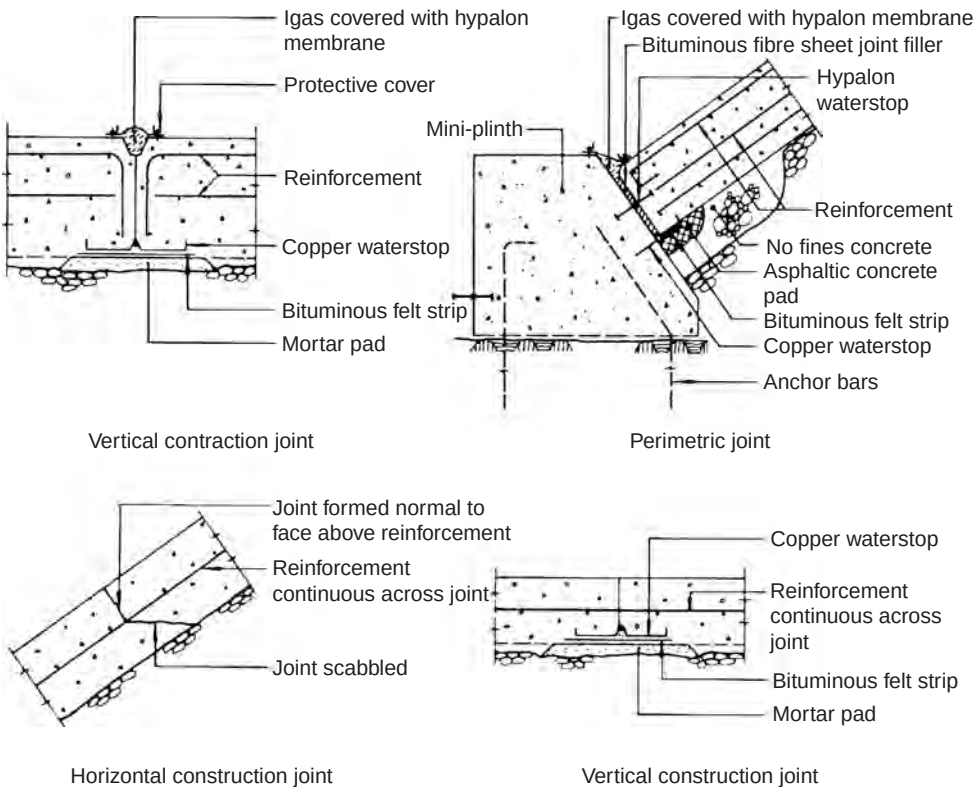


Figure 15.16 Joint details for Khao Laem Dam (Watakeekul *et al.*, 1985).

with a compressible filler, as these have been shown to compress under load and cause opening of the perimetric joint.

Cooke and Sherard (1987) advocated carrying the horizontal reinforcement through all but a few joints near the abutments, in the manner shown in Figure 15.16. This method gives a cost advantage and a reduction in potential leakage, by potentially eliminating water stops and anti-spalling steel. The method may also have contributed to the face cracking at Khao Laem (Materon and Mori, 2000). This is discussed in Section 15.6.

For Aguamilpa dam and many more recent dams, different details were used, depending on whether the joints were expected to be in tension or compression as shown in Figure 15.18. Anti-spalling reinforcement is provided in highly stressed central vertical joints to avoid failure like that at Campos Novos dam shown in Figure 15.20.

Cruz *et al.* (2009) report that following the damage to the face slab on Barra Grande, Campos Novos and Mohale dams a compressive filler of wood or equivalent

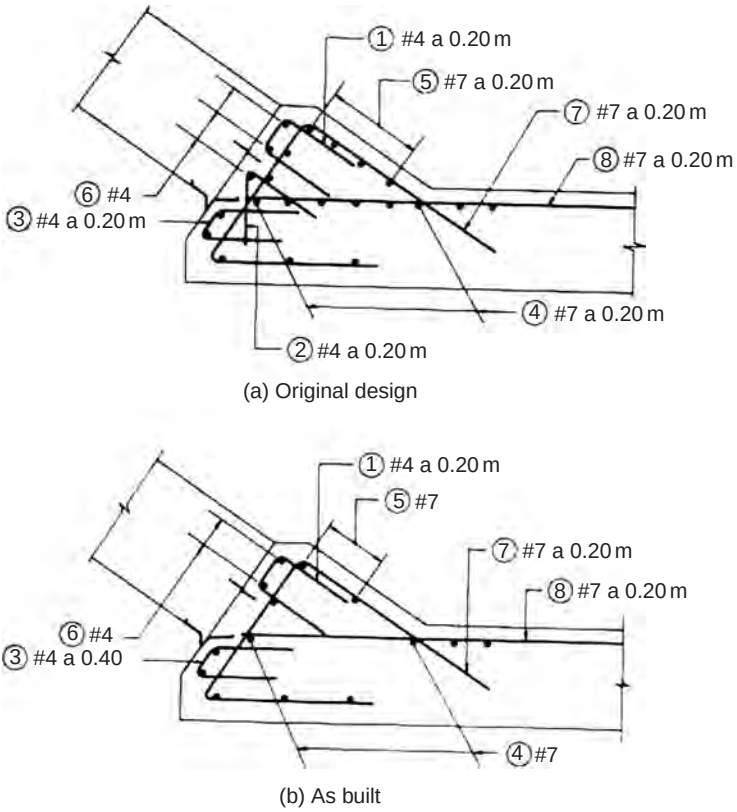


Figure 15.17 Anti-spalling steel in Salvajina Dam (Hacelas and Ramirez, 1987).

material has been used to absorb and distribute compressive stresses on several high CFRD and CFRD in narrow valleys.

15.3.3 Perimetric joint

15.3.3.1 *General requirements*

Instrumentation of CFRDs has indicated that compressive strains develop in more than 90% of the face slab due to settlement of the rockfill.

When the reservoir is filled there is further displacement of the face slab, which leads to closing of vertical joints over most of the slab and opening of the perimetric joint and those joints near the abutments. The face slab also pulls away from the plinth, and offsets normal to the face slab, and parallel to the joint due to shear movement of the face. The joint is a common cause of leakage if not well designed, constructed and inspected.

Table 15.12 summarizes maximum perimetric joint movements measured on a number of CFRDs.

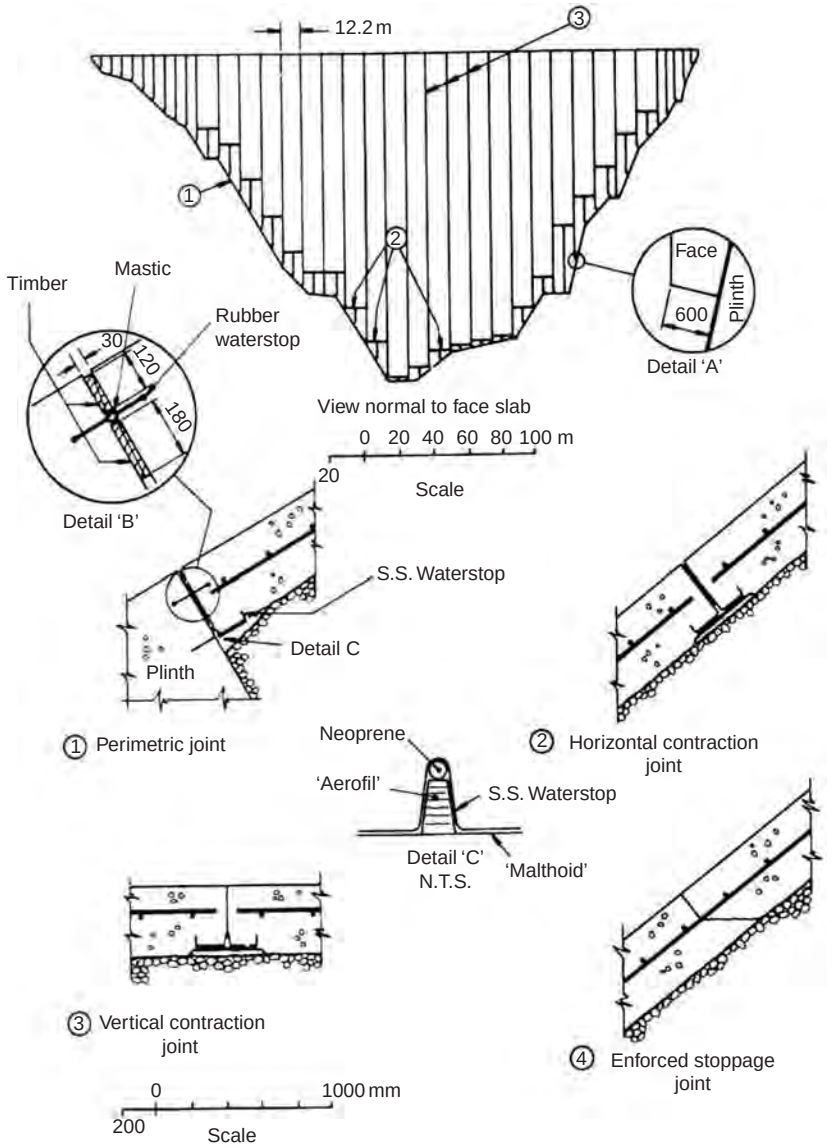


Figure 15.18 Joint layout and details for Reece Dam (Fitzpatrick *et al.*, 1985).

To accommodate these movements, joints with multiple water stops are provided. Figures 15.16, 15.18 to 15.21 show some examples.

The design shown in Figure 15.18 for Reece Dam is typical of practice up till around the mid 1980s. The joint includes two water stops:

– Primary – copper or stainless steel 'W' or 'F' shaped.
– Secondary – central 'bulb' water stop made of rubber, hypalon or PVC.

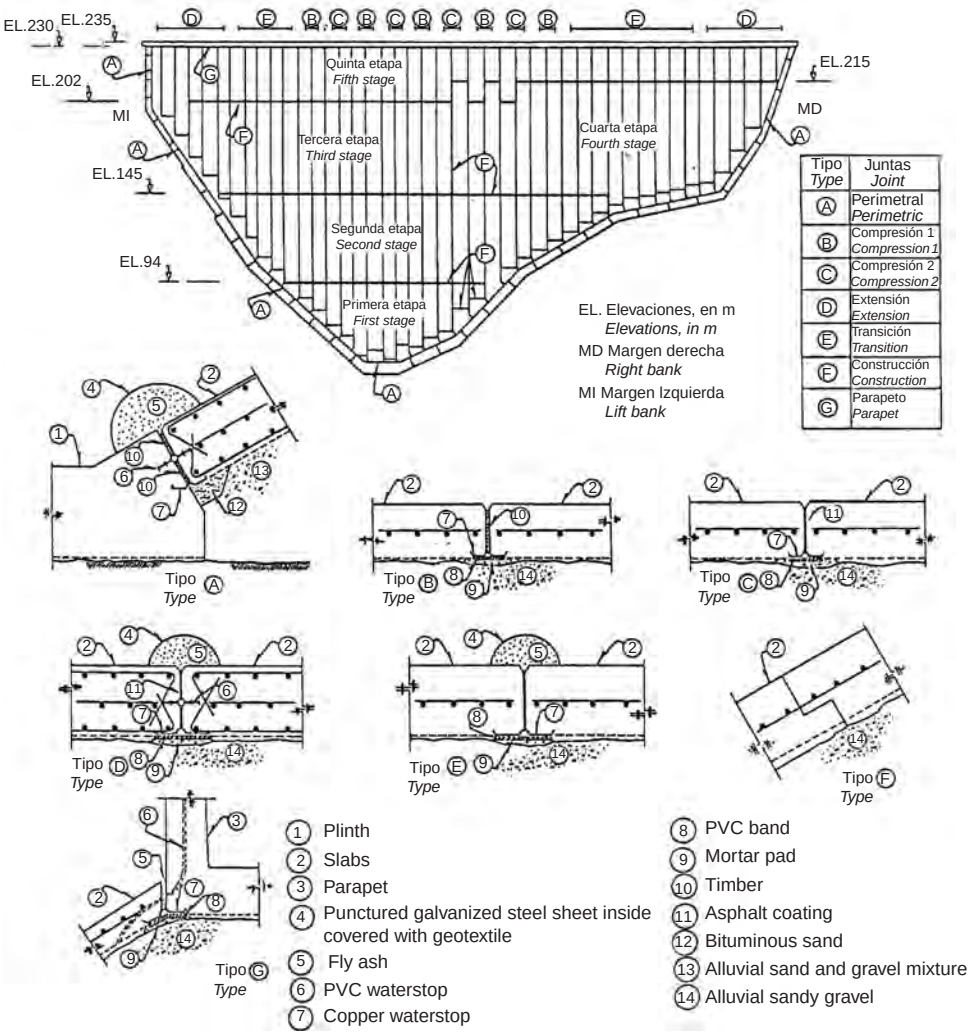


Figure 15.19 Joint details for Aguamilpa dam (a) elevation showing joint layout, (b) joint details (Valencia and Sandoval, 1997).

Cooke and Sherard (1987) indicate that this arrangement has performed adequately on dams up to 75 m high, where perimetric joint movements are generally relatively small. Fitzpatrick *et al.* (1985) indicate that in two dams, 39 m and 26 m high, only the primary water stop was used because very small movements were expected.

More recently, particularly for higher dams, a third water stop has been included in the form of mastic or fly ash filler covered with a PVC or hypalon sheet. These are shown in Figures 15.16, 15.19, 15.21 and 15.22 for Khao Laem, Aguamilpa, Salvajina and Antamina dams. Figure 15.23 shows two more recent designs. Each involves multiple lines of defence.



Figure 15.20 Campos Novos Dam. Central compressive vertical joint failure (Cruz *et al.*, 2009).

Cooke and Sherard (1987) argued that, because of the difficulty of having concrete around the central 'bulb' water stop, they believe that a joint with the copper (or stainless steel) water stop underlain by asphalt impregnated sand or concrete mortar and the mastic type stop, is the preferable detail. For Antamina dam (Amaya and Marulanda, 2000) the intermediate PVC seal was eliminated and a 1 m thick sand zone (2A) placed under the perimeter joint. These modifications were incorporated to ease construction and control potential seepage in joints that have performed well in practice. ICOLD (2010) give more information.

It will be noted that in the designs shown, a wood plank approximately 12.5 mm to 20 mm thick, or some other compressible filler of similar thickness, is placed between the face slab and plinth to prevent concentration of stresses in the joint during construction and before reservoir filling.

15.3.3.2 *Water stop details*

Primary copper or stainless steel water stop. These are either 'W' or 'F' shaped, with a high central rib to permit shear movement between adjacent slabs. To prevent external water pressure from squeezing the rib flat, it is filled with a neoprene insert, 12 mm diameter, held in place with a strip of closed cell polythene foam 16 × 12 mm (Fitzpatrick *et al.*, 1985). The water stop is supported on a cement mortar or asphalt impregnated sand pad.

Whether copper or stainless steel is used depends on the aggressive nature of the reservoir water, but also seems to be a matter of individual designer preference, with copper being more common. Fitzpatrick *et al.* (1985) indicate that for the Reece Dam the HEC departed from its earlier practice of using copper water stop (annealed after forming to give maximum ductility) and used 0.9 mm thick grade 321 stainless steel. This was done because it was considered that the stainless steel would be 'more robust' during construction, and there was not a significant cost differential, and the stainless steel would be less affected by the acid reservoir water.

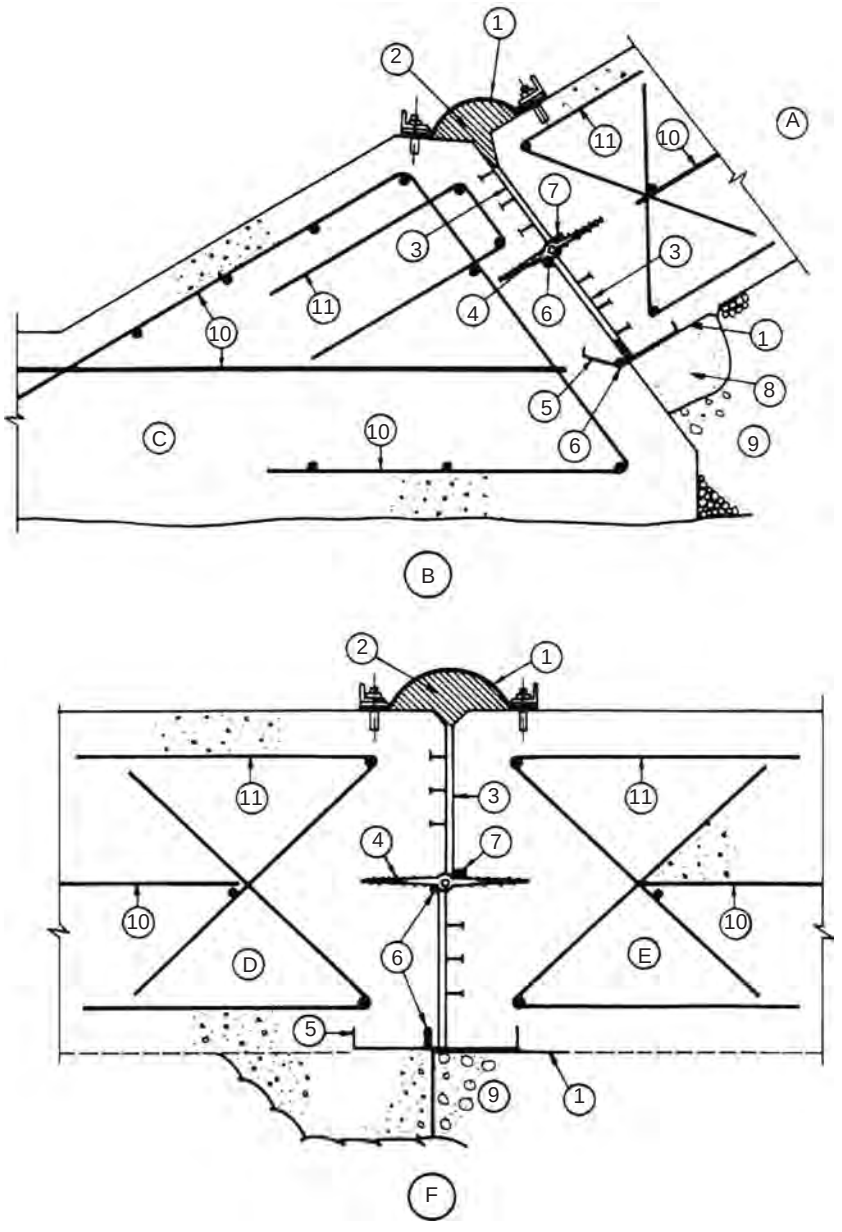


Figure 15.21 Salvajina Dam perimeter and near abutment vertical joints (ICOLD, 1989a). (1) Hypalon band, (2) Mastic filler, (3) Compressible wood filler, (4) PVC waterstop, (5) Copper waterstop, (6) Neoprene cylinder, (7) Styrofoam filler, (8) Sand-asphalt mixture, (9) Zone 2, (10) Steel reinforcement, (11) Steel reinforcement to protect concrete against crushing and to protect waterstop. (A) Face slab, (B) Perimetric joint, (C) Plinth, (D) Plinth, (E) Face slab, (F) Abutments.

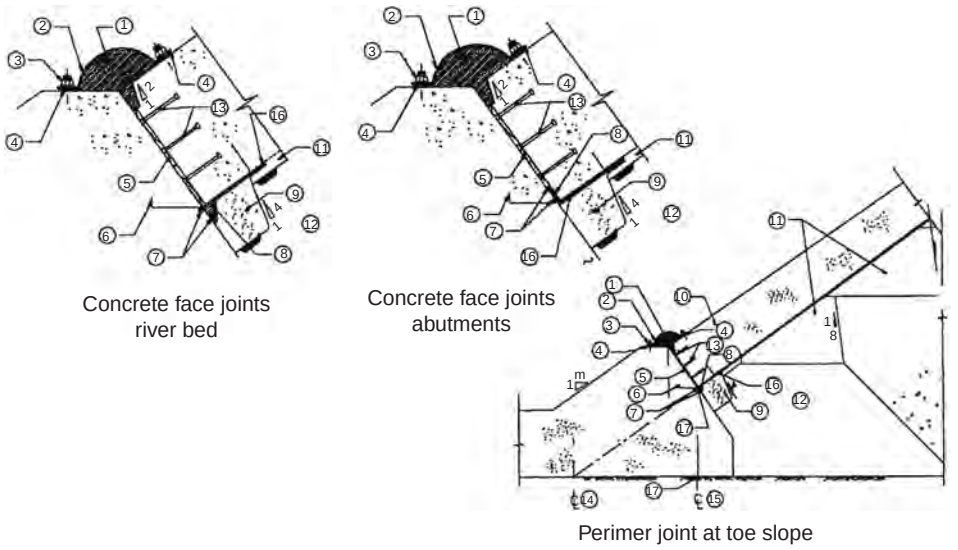


Figure 15.22 Antamina Dam joint details (Amaya and Marulanda, 2000). (1) Fly-ash, (2) PVC band, (3) Anchor bolts, (4) Epoxy glue, (5) Abarco wood, (6) Copper seal, (7) Circular neoprene, (8) Styrofoam, (9) Sand/asphalt mix, (10) Slab, (11) concrete curb, (12) Zone 2A, (13) Steel bolts, (14) Plinth excavation reference line, (15) Plinth construction reference line, (16) PVC strip.

ICOLD (1989a) indicate that it is advisable to form the copper or steel water stops in continuous strips to minimize the need for field splices. They recommend use of an electrode of high fluidity (silver content greater than 50%) for welding copper water stops to ensure full penetration into the two copper plates, then checking with a spark tester to ensure a good joint has been achieved. Fitzpatrick *et al.* (1985) indicate that for stainless steel, jointing consists of a lap joint fixed by spot welding, then sealing by tungsten-inert gas welding, so that only one metal is involved.

Pinkerton *et al.* (1985) indicate that in their experience the limit of shear displacement of 'W' type primary water stops is around 7.5 mm compared to up to 50 mm for a 230 mm centre bulb rubber water stop and 10 mm for a PVC water stop.

ICOLD (2010) give more details on these types of water stops.

Centre bulb water stop. These are constructed of PVC, natural rubber or hypalon. Fitzpatrick *et al.* (1985) indicate that they prefer hypalon rubber instead of natural rubber or PVC because natural rubber in the atmosphere must be protected from oxidation and ozonation by the addition of antioxidants and anti-ozonants which could leach out. These materials will last indefinitely below minimum operating level where permanently submerged, but there may be a problem between minimum operating level and flood level as PVC contains plasticizers, some of which are known to leach out.

Pinkerton *et al.* (1985) indicate a preference for hypalon rubber because it can accept much larger deformations as detailed above.

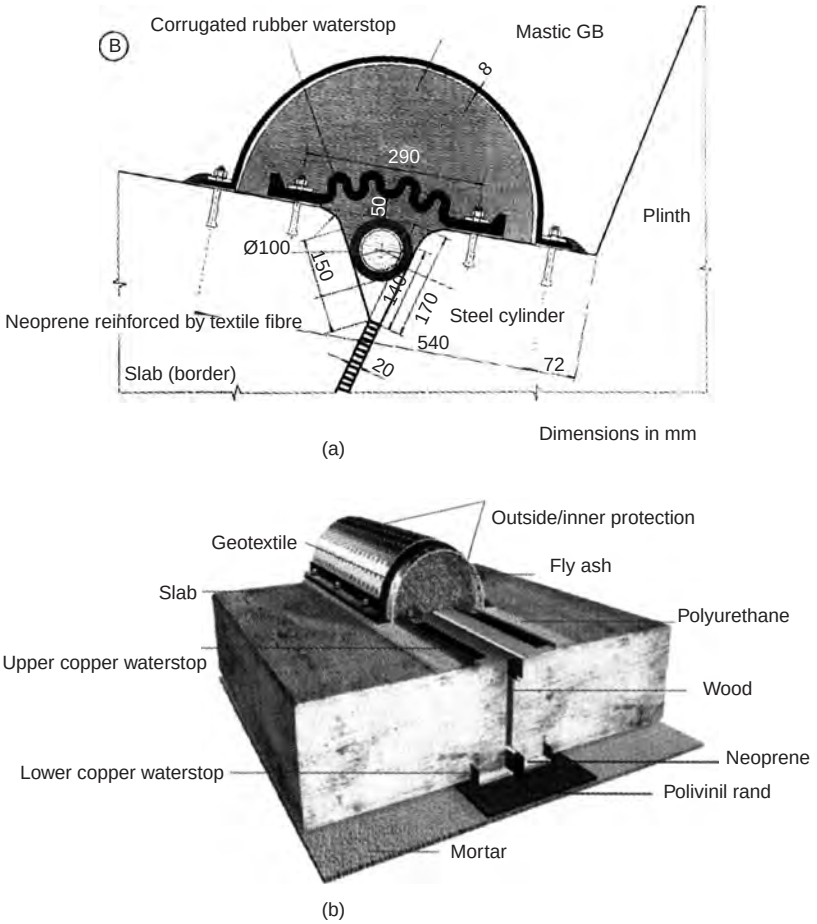


Figure 15.23 Perimeter joint details for (a) Mazar Dam; (b) El Cajon Dam (Cruz *et al.*, 2009).

As mentioned above, in the design of Antamina dam (Amaya and Marulanda, 2000) the central PVC seal was eliminated.

Mastic filler water stop. The concept of the mastic filler is that, as the perimetric joint opens, it will be forced into the opening by the water pressure. The mastic is covered with a PVC or hypalon membrane held in place by steel angles anchored to the concrete.

ICOLD (1989a) indicate that a chicken wire mesh was embedded in the mastic to prevent its flow downwards along the inclined joints. The covering membrane is convex upward to provide for enough mastic volume. It is important that the membrane be sealed effectively so that the water pressure does not leak past the membrane, relieving the differential pressure needed to force the mastic into the crack. Adhesion is improved by painting the joints with mastic.

Cooke and Sherard (1987) indicate a preference for the membrane to be hypalon, not PVC (at least for higher dams), because of its proven 10 to 20 year life exposed to

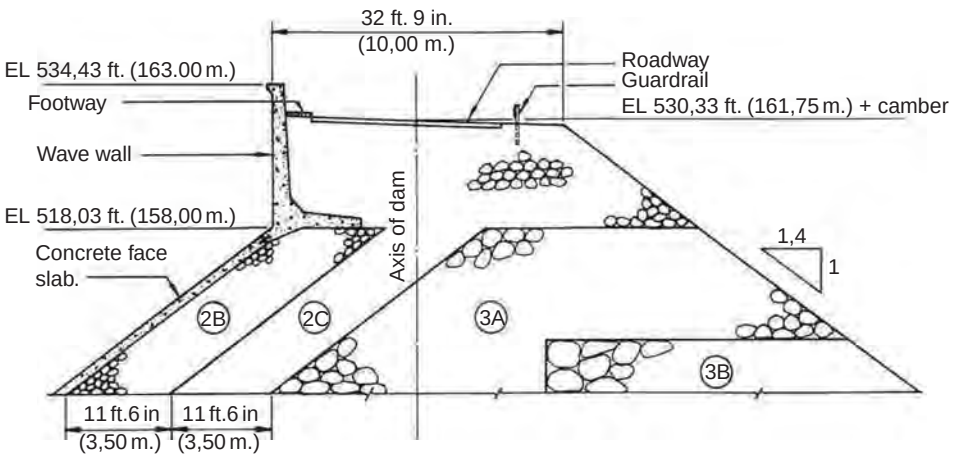


Figure 15.24 Crest detail, Khao Laem Dam (Watakeekul *et al.*, 1985).

the weather. After this period the cover is not critical, because joint opening should have ceased, and the mastic is wedged tightly into the joint, stopped against the other water stops or, if they have ruptured, against the mortar or asphalt impregnated sand pad.

The mastic which has been commonly used is IGAS which is a bitumen compound. It retains its flow characteristics provided it is not exposed to sunlight for extended periods. This did occur in Golillas Dam (Amaya and Marulanda, 1985) where the PVC and IGAS were exposed for about four years before the reservoir was filled.

Some recent dams have replaced the mastic with ash or sand, which is to fill the joint opening, and be retained by the other water-stops or, in the event they fail, by the Zone 2A filter beneath. Figure 15.17 and Figure 15.18 show such details. Cruz *et al.* (2009) give other examples.

It should be recognized that older dams which only had one water stop; e.g. a lower copper water stop; are more vulnerable to leakage at the perimetric joint than more modern designs which have multiple water stops. These dams are more likely to experience leaks in the event the water stop is ruptured under seismic or high reservoir level loads. This issue is especially a worry where the perimetric joint, most likely near the crest of the dam, is along a vertical wall face so that there may be quite a significant depth of fill below and hence a real chance of settlement to occur below the face slab at the joint.

15.3.4 Crest detail

It is common to provide a reinforced concrete retaining wall ("wave-wall", "crest wall" or "parapet wall") at the crest of the dam to reduce the volume of rockfill. Wave walls up to 3 m to 5 m have been used.

Figure 15.24 and Figure 15.25 show details which have been adopted for Khao Laem, Golillas Dam and Macaqua Dam.

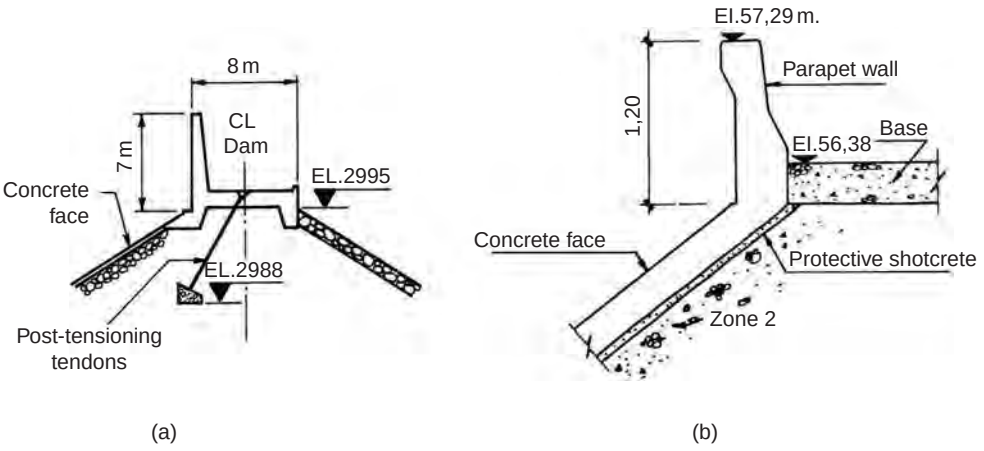


Figure 15.25 Crest details for (a) Golillas Dam, (b) Macaqua Dam (Amaya and Marulanda, 1985 and Prusza *et al.*, 1985).

The base of the wall is usually above full supply level and the wall is joined to the face slab with a flexible joint, such as that shown in Figure 15.26. The joint should be vertical, not normal to the plane of the slab, so that differential settlement can be accommodated.

The crest width depends on operational requirements but may be as narrow as 4.9 m. The crest wall is constructed after the face slab, giving a relatively wide platform on which to work while the face slab is under construction.

At Gouhou Dam (Chen, 1993) embankment failure occurred on first filling when the water level rose above the joint between the face slab and the crest wall (Figure 15.27). This joint had opened as a result of settlement of the slab but poor concrete construction appears to have been a contributing factor. As discussed in Section 15.2.2.3, the critical issue was the absence of a free draining zone in the dam to discharge the leakage.

15.4 CONSTRUCTION ASPECTS

15.4.1 Plinth construction and special details

The methods of constructing the plinth are related to the topography and geology (Materon and Mori, 2000). In wide valleys, e.g. Xingo, Foz do Areia, and Ita, plinth construction can be established independently of embankment placement. In narrow and steep valleys, e.g. Alto Anchicaya and Golillas, plinth excavation was simultaneous with the fill construction.

It is desirable for the plinth to be founded on sound, non-erodible, groutable rock. In good rock requiring blasting, excavation is commonly controlled by pre-splitting with holes at 0.6 m spacing. Dental concrete is applied before anchor installation. In weathered rock blasting should be minimized with excavation by machine or even

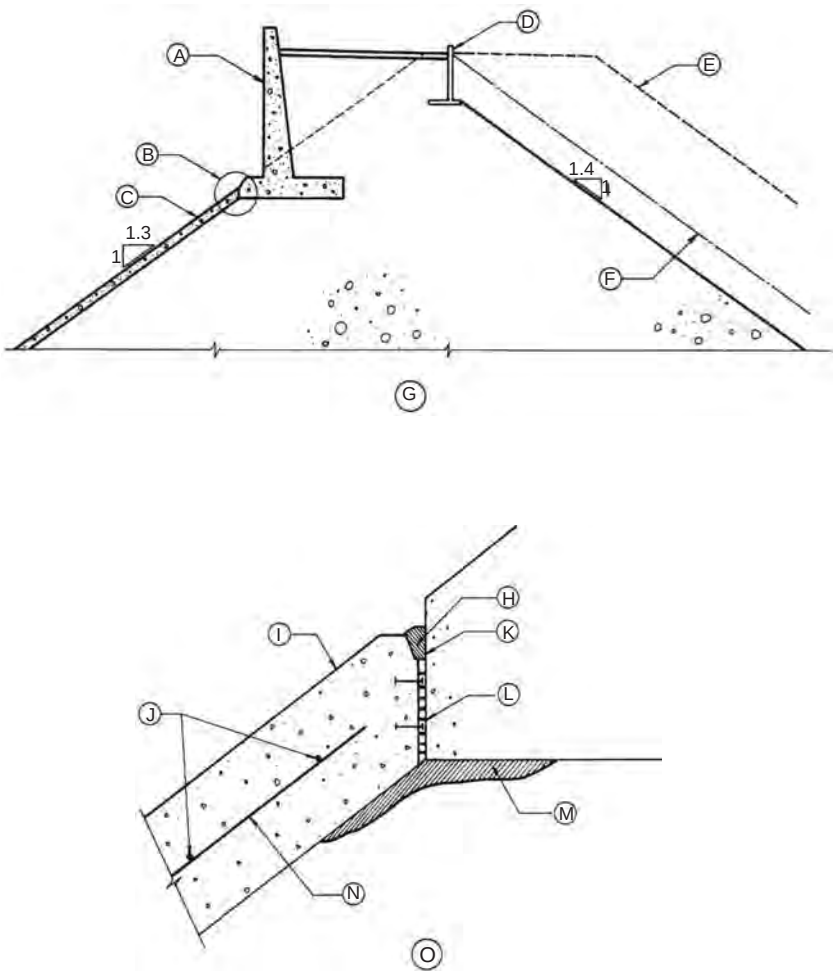


Figure 15.26 Crest details (ICOLD, 1989a). (A) Upstream parapet wall, (B) See detail, (C) Concrete face, (D) Downstream parapet wall, (E) Additional rockfill if both parapet walls are omitted, (F) Additional rockfill if downstream parapet wall is omitted, (G) Crest details, (H) IGAS, (I) Concrete face, (J) Horizontal reinforcement, (K) IGAS primer, (L) Pre-moulded filler, (M) Mortar pad, (N) Inclined reinforcement, (O) Detail: concrete face and upstream parapet wall joint.

by hand. Figure 15.28 illustrates design features for Salvajina Dam (Marulanda and Pinto, 2000) where special foundation treatment included provision of a sand filter and gravel transition beneath the upstream section of the embankment, consolidation grouting beneath the plinth, removal of altered material to a depth of 3–4 times the seam width and backfill with concrete, protection of exposed slopes upstream of the plinth with steel reinforced shotcrete.

CFRDs built on alluvium, e.g. Santa Juana and Puclaro in Chile and Punta Negra and Los Caracoles in Argentina; involve the construction of an articulated plinth on

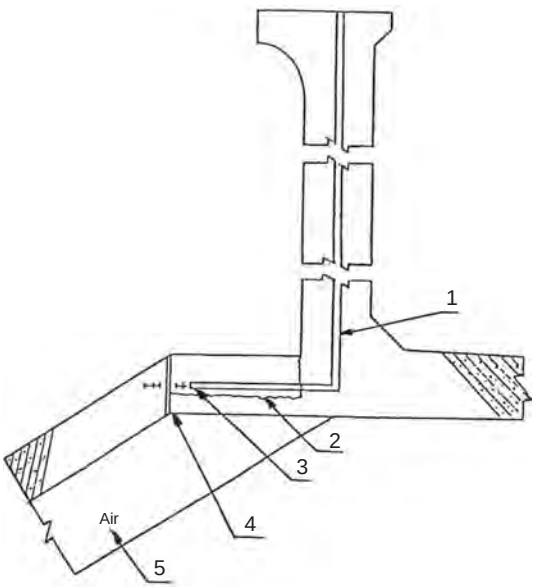


Figure 15.27 Gouhou Dam – Crest wall details (Chen, 1993). 1. Rubber water stop; 2. Top of the intact concrete; 3. Rubber water stop; 4. Timber Fill; 5. Transition.

compacted gravel with joints protected by a transition filter. The plinth is connected to a concrete diaphragm wall taken through the alluvium. Figure 15.29 shows the plinth and cut-off wall at San Juana (Marulanda and Pinto, 2000).

The plinth is constructed using concrete with strength of 21 MPa at 28 days using metal or timber forms. This is followed by grouting of the foundation either downstage or upstage.

15.4.2 River diversion

Generally the river is diverted by tunnels but many projects have allowed for overtopping of the rockfill embankments provided protection measures to prevent unravelling of the downstream face are taken. These measures include steel reinforcement of the rockfill and gabions anchored into the rockfill. In large rivers a diversion strategy is chosen to optimize the tunnel size, incorporating a "priority" section of the CFRD which has a low probability of being overtopped (Figure 15.30).

15.4.3 Embankment construction

Zones 2D and 2E

Zones 2D and 2E are commonly placed in 0.2 m to 0.5 m layers with Zones 3A and 3B from 0.8 m to 2.0 m thick (Materon and Mori, 2000). Zone 2D is commonly compacted using manual vibratory compactors. In some projects 3–4% of cement was added to give some cohesion. (Note. The authors would not recommend this because

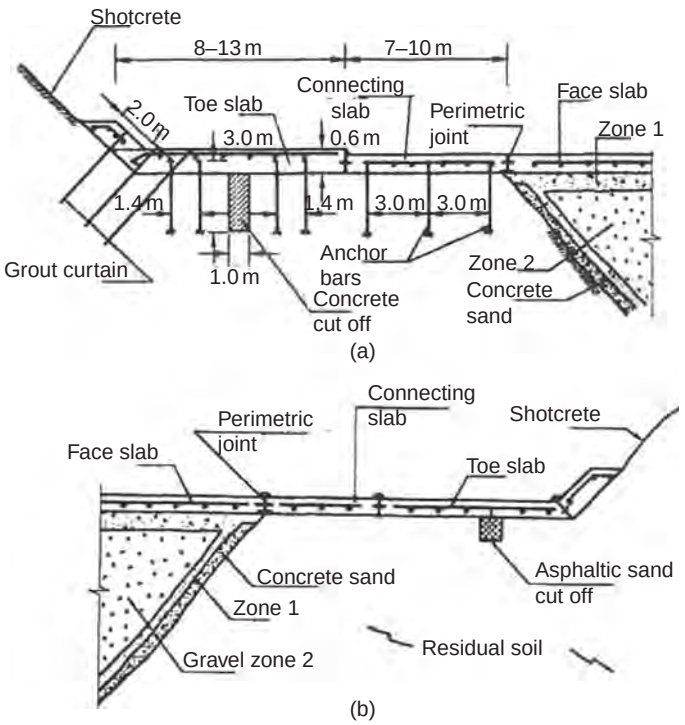


Figure 15.28 Foundation treatment and plinth design Salvajina Dam (Marulanda and Pinto, 2000). (a) Plinth on less competent rock (b) Plinth on residual soil.

it would restrict the ability of Zone 2D to self-heal against Zone 2E in the event of a leak). Zone 2E is compacted with a few passes of a 5 tonne non vibrating roller.

The face is protected either by spraying with an asphaltic emulsion or forming a concrete kerb. Shotcrete has been used instead of bitumen emulsion; e.g. on Golillas and Salvajina dams 40 mm of shotcrete with size 9 mm to 19 mm in maximum size was adopted.

At Altamina Dam (Amaya and Marulanda, 2000), Ita dam in Brazil (Sobrinho *et al.*, 2000) and more recently in several dams an extruded concrete kerb 0.40 m high was constructed at the upstream edge of each fill layer to allow placement of Zone 2D without having to compact it on the sloping face and to limit erosion of the upstream face prior to constructing the face slab. Figures 15.31 and 15.32 show construction of the kerb, and the sequence of construction.

This is economical and allows higher construction rate. The kerbs are not a replacement for the face slab, just a formwork for placing Zone 2D. They are coated on the upstream face with bitumen to prevent bond and tensions on the face slab. Construction methods for the concrete kerb are described in Materon and Mori (2000).

The authors have some concerns about the use of kerbs because no matter what is done there will be some shear transfer between them and the face slab making the

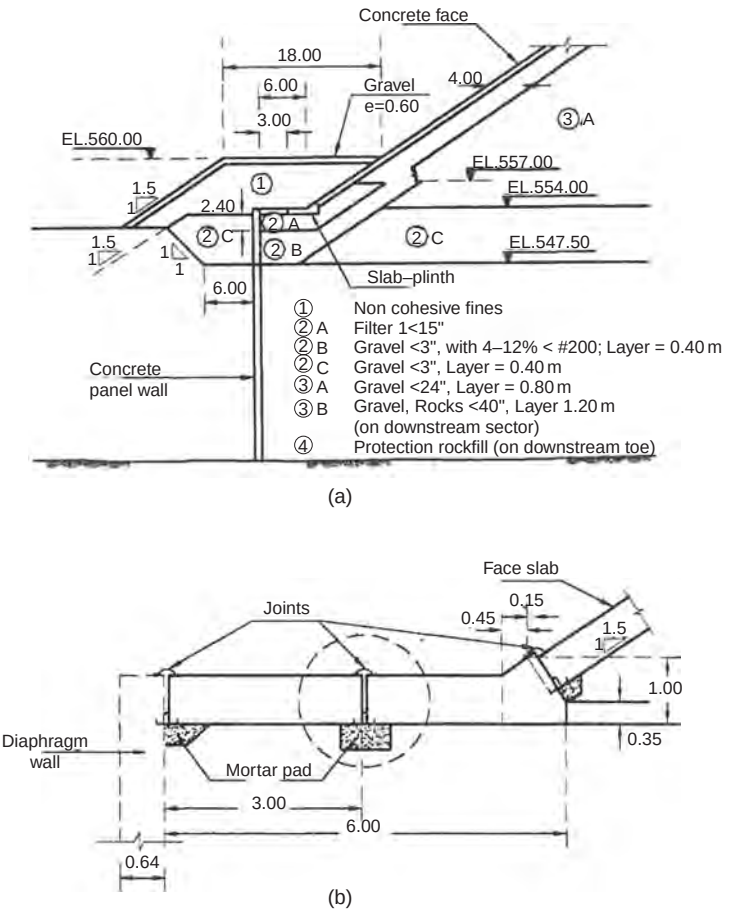


Figure 15.29 Foundation treatment and plinth design San Juana Dam (Marulanda and Pinto, 2000).

face slab thicker than designed for. This may lead to cracking from below when the water load is applied.

Zones 3, 3A, 3B

Zones 3, 3A and 3B rockfill are placed using truck and dozer in layers from 0.8–1.0 m in Zone 3A and 1.6–2.0 m in zone 3B and compacted with vibratory steel drum rollers. Zones 3A and 3B using gravel are built in a similar way with smaller layer thicknesses of 0.6–1.2 m (Materon and Mori, 2000). Water is added as discussed in Section 15.2.3.

Face slab construction

The concrete face slab is cast using a slip-form, except for the trapezoidal or triangular starter slabs adjacent to the face slabs, which are screeded by hand methods ahead of the main face slab to provide a starting plane for the slip-form. These are usually half the width of the main slabs.

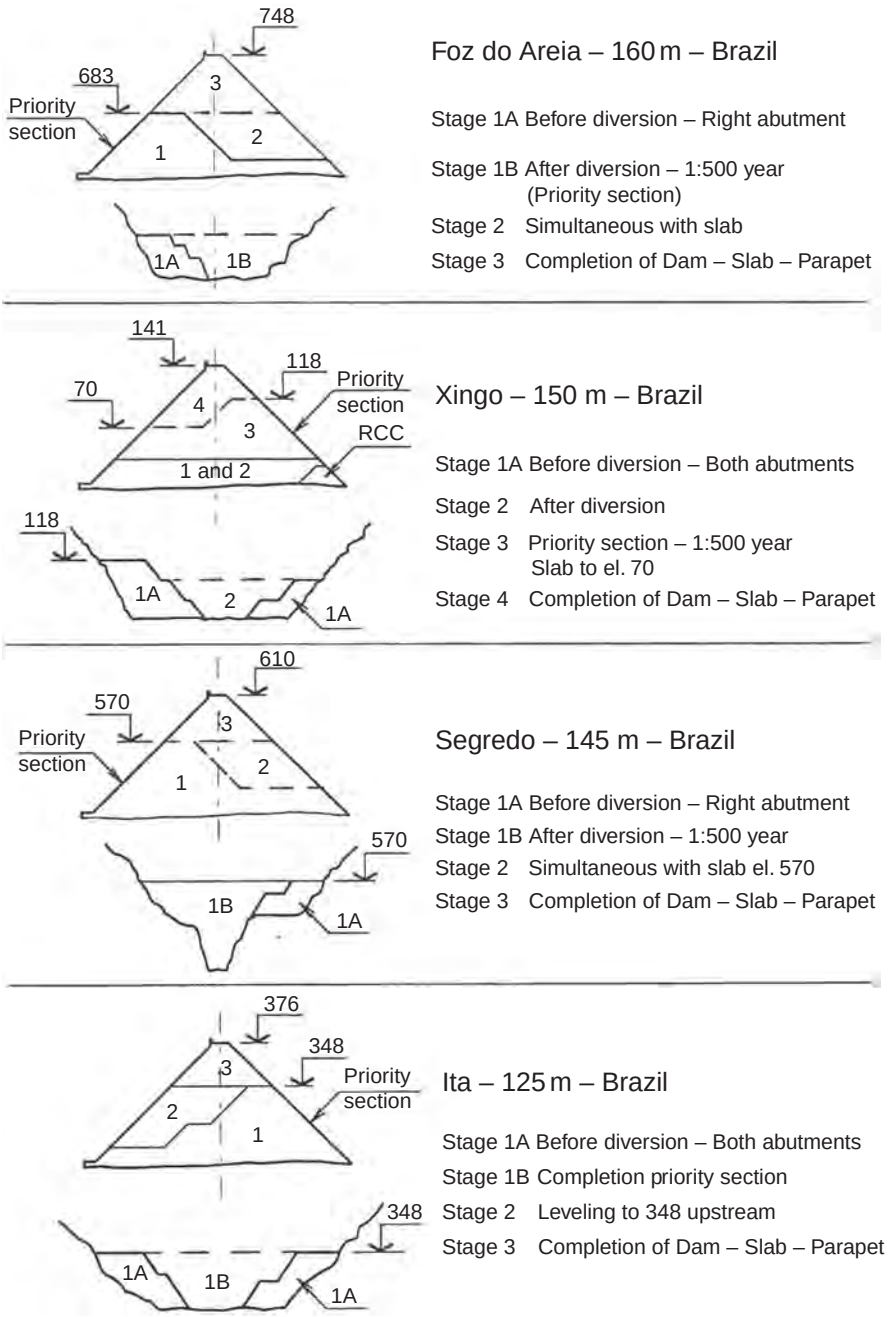


Figure 15.30 River diversion stages in CFRD's (Materon and Mori, 2000).



Figure 15.31 Extruded kerb construction for the upstream face (Cruz *et al.*, 2009).

The slip-form is the full width of the panel. For a 1.3 m screed width, the form can move at 2 m to 3 m per hour placing 60 mm slump concrete (Cooke, 1984). Higher slump concrete requires a wider screed. Figure 15.33 shows the slip form used for Khao Laem Dam.

Varty *et al.* (1985) give details of steel placement and other construction factors, as practiced on HEC dams.

Concrete is delivered to the form by bucket, pumping or in a chute.

The concrete used is typically specified as between 20 MPa and 24 MPa at 28 days, (ICOLD, 1989a), although higher strengths are used, e.g. Cethana and Lower Pieman had average 40 MPa concrete (Fitzpatrick *et al.*, 1985). (ICOLD, 1989a) indicate that higher strengths are not desirable as more shrinkage cracking is likely. They suggest the use of air entrainment to enhance water tightness and durability. The soundness and reactivity of aggregates used for the concrete is important. Materon and Mori (2000) give some more recent experiences which are similar to those described above.

The face slab is constructed on the face as it is presented. The face may have moved subsequent to trimming and compaction, due to the continual raising of the dam. The slab may therefore not end up being on a single plane, but the small variations do not affect performance or appearance.

The design thicknesses are minimum values, and average thicknesses are likely to be 50 mm to 75 mm greater. Early practice was to require that the face slab not be constructed until the rockfill placing was virtually completed so as to minimise post construction movements. More recently it has been shown that staging of the concrete face slab is acceptable, e.g. Salvajina and Foz do Aeria dams (Sierra *et al.*, 1985 and Pinto *et al.*, 1985), but cracking may occur if the zones have differential moduli leading to localisation of strains. If the face slab is constructed in stages, a common practice now for very high dams, or if an existing dam is to be raised, some cement grouting

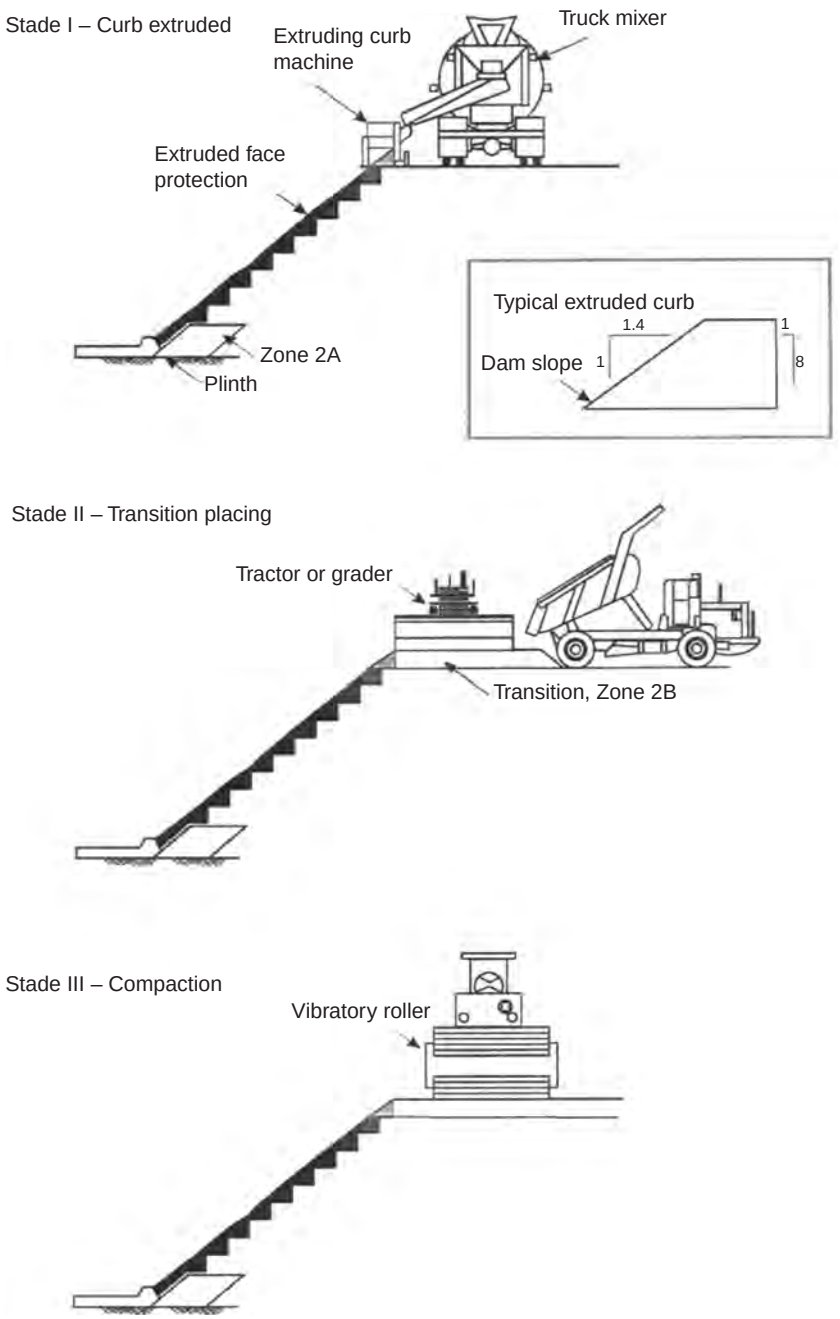
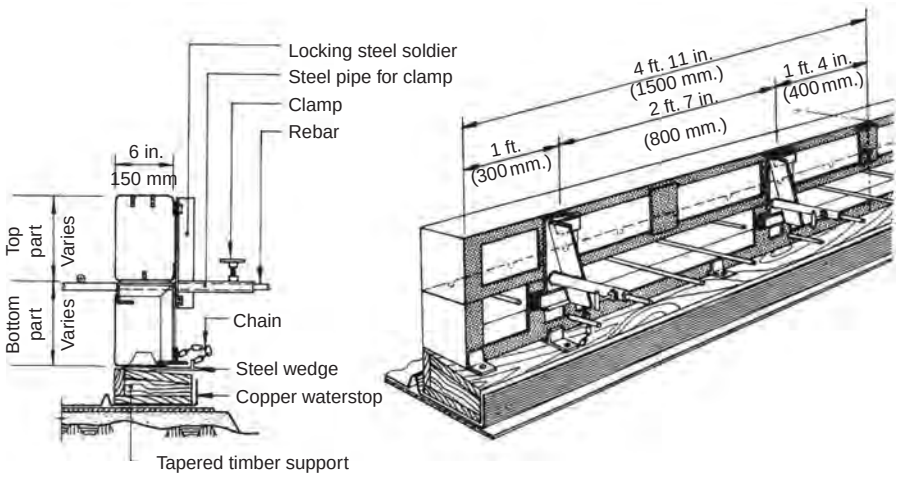
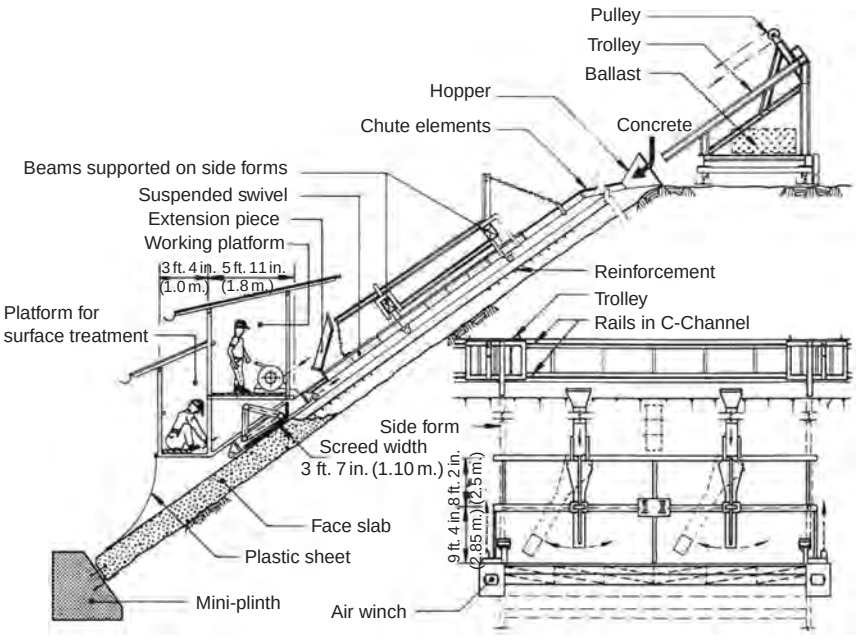


Figure 15.32 Sequence of Kerb construction and placement of Zone 2D (shown as Zone 2B in figure) (ICOLD, 2010, from Resende and Materon, 2000).

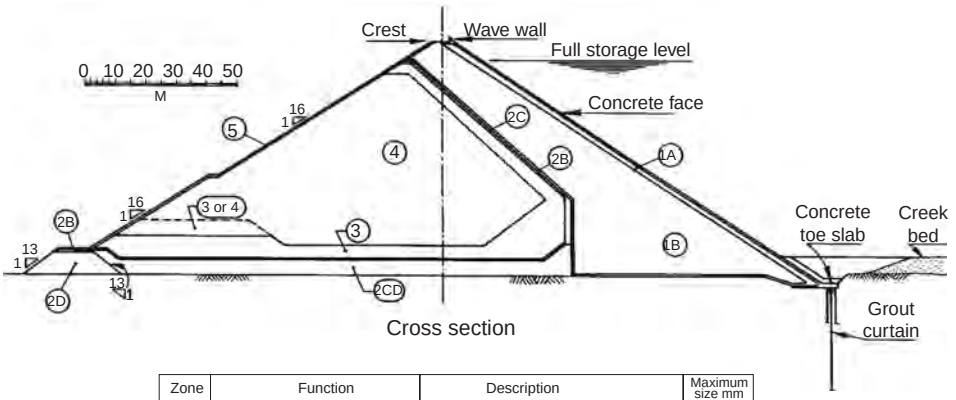


Section Perspective



Section Plan

Figure 15.33 Slip-form for Khao Laem Dam face slab (Watakeekul *et al.*, 1985).



Zone	Function	Description	Maximum size mm
1A	Semi-pervious zone	Fresh siltstone	150
1B		Fresh sandstone and siltstone	450
2A	Filters	Fresh basalt	20
2B			75
2C	Chimney drain		300
2CD	Bottom drain		600
2D	Toe drain	Fresh basalt from 100mm to 1200m	1200
3	Free draining layer	Fresh sandstone and/or siltstone	600
4	Random zone	Fresh or weathered sandstone and siltstone	450
5	Downstream slope protection	Fresh sandstone from 600mm to 1200mm	1200

Figure 15.34 Mangrove Creek Dam (MacKenzie and McDonald, 1985).

infill may be required beneath the top of the already placed slab. The fill above may cause a localised void to open up behind the top of the slab.

15.5 SOME NON-STANDARD DESIGN FEATURES

15.5.1 Use of dirty rockfill

When the available rockfill breaks down under compaction to form a rockfill with a large proportion of sand and silt/clay size particles, the resulting fill may have adequate modulus, but may not be free draining. Rocks which are likely to do this are sandstones, siltstones, shale, schists and phyllites.

If this is the case, CFRD can still be used but must be zoned to provide drainage layers behind the face slab and on the foundation.

Mangrove Creek Dam is an example of this design, as discussed in detail in MacKenzie and McDonald (1985).

Figure 15.34 shows the zoning.

Other examples are Kangaroo Creek and Little Para Dams (Good *et al.*, 1985) and Salvajina Dam (Sierra *et al.*, 1985). Zoning for Salvajina Dam is shown in Figure 15.7. In this case the rockfill zones are compacted gravel which has significant silt content, and the drainage layer 2A is provided to ensure dam leakage will not result in build up of pore pressure in the dam.

The downfall with this approach might be that the Zone 2A must have an adequate vertical drainage capacity. If there has been a lot a breakdown at the surface of each layer of fill, which have been reported at some CFRDs, the drainage capacity may not be adequate under certain circumstances, such as serious face slab and perimeter damage during a large earthquake.

It is not uncommon to zone the rockfill to ensure that the most free draining material is downstream of Zone 2E and on the base of the dam.

15.5.2 Dams on erodible foundation

Concrete face rockfill dams have been successfully constructed on foundations which have weathered seams or clay filled joints, which were potentially erodible under the high gradients which occur under the face slab.

In these circumstances, it has been necessary to prevent erosion by lengthening the seepage path and by providing filters into which the under-seepage emerges in a controlled manner.

An example is Winneke Dam. The deeply weathered siltstone rock precluded economic placement of the plinths on fresh rock and the slab was founded at a level where weathered rock contained infilled joints and crushed seams, some of which were dispersive clays (Figure 15.35).

The measures adopted are described in Casinader and Watt (1985), Casinader and Stapledon (1979) and Stapledon and Casinader (1977) and included:

- The upstream toe excavation was taken down to the top of the highly weathered rock zone to reduce the number of infilled seams beneath the plinth.
- Prior to grouting, seams in the grout holes were flushed out as far as practicable using air and water.
- The plinth width adopted was 0.1H, or 6 m minimum, where H is the water head – wider than normally adopted to reduce gradients. The foundation downstream of the plinth was blanketed with 150 mm of concrete, so that the total width of the plinth and 'foundation concrete' was at least 0.5H.
- A filter was placed over the foundation for a distance 0.5H downstream of the foundation concrete and the filter and foundation concrete was covered by the transition layer to control erosion of fines from the foundation (even if the foundation concrete cracked).

The possibility of sliding movements on weak seams also necessitated a buttress on the plinth, as shown in Figure 15.35.

Reece Dam was founded on deeply weathered schists with clay seams at cutoff level on the left abutment. In this case, the abutment downstream of the plinth was covered with a 150 mm layer of steel meshed shotcrete for a distance 0.5H downstream. The whole shotcrete layer, and the weathered rock between it and the downstream toe, was then covered by 2A and 2B type filter materials.

At Mohale Dam (Gratwick *et al.*, 2000) two significant geological lineaments cross the dam site and were assessed to have potential for movement. Treatment included the excavation of a slot to provide a concrete 'socle' 15 m wide and 2 m deep below the toe-slab, a grout curtain directed through the lineament and a grouting

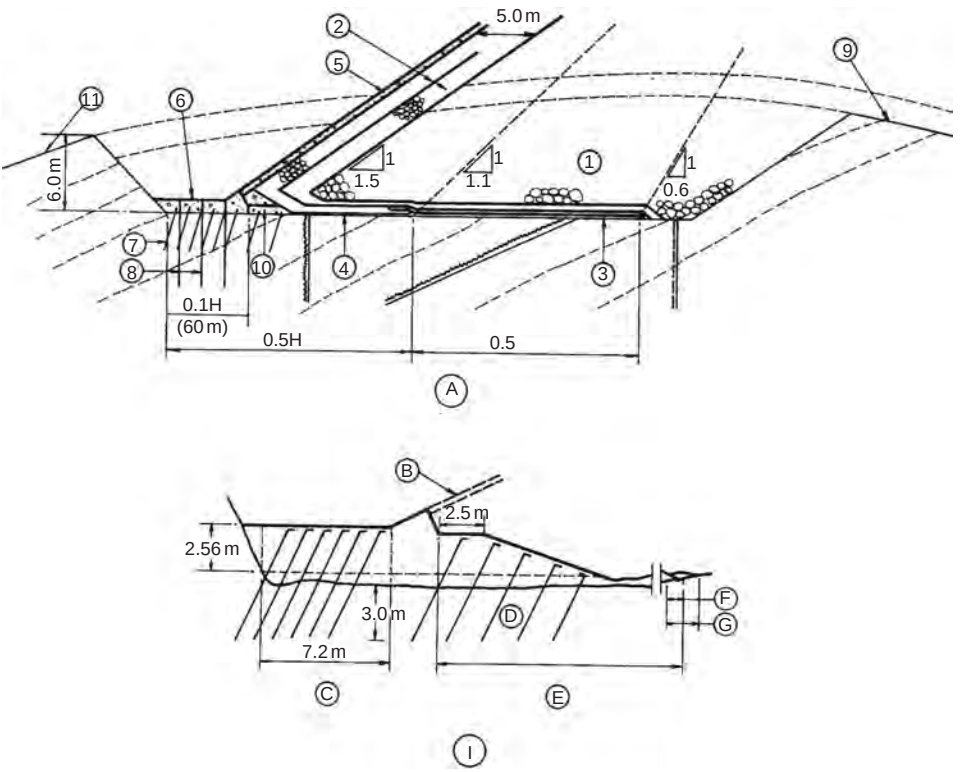


Figure 15.35 Winneke Dam foundation treatment (ICOLD, 1989a and Casinader and Watt, 1985). (A) Upstream toe detail, (B) Concrete face, (C) Anchor bars at 1 m centres longitudinally, (D) Foundation concrete 0.15 m thick, width 32.25 m, (E) Anchor bars at 2 m centres each way within thickened section, (F) Overlap 2 m minimum, (G) Fine filter 42–30 m, (H) Hydraulic head at foundation level, (I) High plinth with buttress. (1) Rockfill, (2) Transition, (3) Filters, (4) Foundation concrete, (5) Concrete face, (6) Plinth, (7) Anchor bars, (8) Grout curtain, (9) Foundation, (10), Buttress, (11) Original ground surface.

gallery extending from the downstream side of the embankment to the toe-slab area near the lineament. Both the "socle" and the toe-slab incorporate movement joints composed of 20 mm of compressible material and a PVC water stop. Upstream of the toe-slab a blanket of earthfill was placed for a distance of 70 m and downstream protection provided by a geotextile and reverse filters of 2D and 2E material.

Another example of special foundation treatment is described by Sierra *et al.* (1985) for Salvajina Dam (Figure 15.28). In this case, the foundation was in part founded on residual soil and wide concrete slabs were added upstream of the plinth and filters downstream.

15.5.3 Leaving alluvium in the dam foundation

When sandy gravel alluvium or glacial soils are present in the river bed, these can commonly be left in place except for the first 0.3H to 0.5H downstream of the plinth.

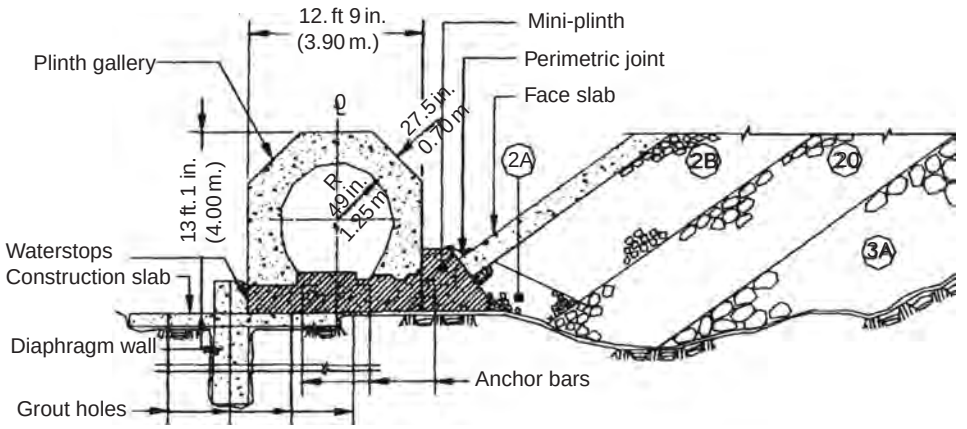


Figure 15.36 Plinth and gallery, Khao Laem Dam (Watakeekul *et al.*, 1985).

This is dependent on confirmation that the gravels are of adequately high modulus and strength. However potentially liquefiable gravels must be removed or densified. This was sometimes overlooked in early CFRD construction.

15.5.4 Plinth gallery

The Khao Laem Dam, which was constructed on deeply weathered and in part karst foundations, incorporated a permanent gallery over the plinth of the dam. Details are shown in Figure 15.36.

The value of the gallery, as a means of access for remedial work, was shown when leakage developed through the perimetric joint. The leak was reduced by grouting from the gallery, as described in Watakeekul *et al.* (1987).

Moreno (1987) describes the incorporation of a grout and drainage gallery in a 190 m high CFRD, in a highly seismic area. Figure 15.37 shows the gallery. Grouting from the gallery assisted in meeting construction schedules.

15.5.5 Earthfill cover over the face slab

ICOLD (1989a), in their drawing showing typical design features of modern CFRDs (reproduced in Figure 15.2), show earthfill and random fill placed over the lower part of the plinth (Zones 1A and 1B).

Cooke and Sherard (1987) point out that these zones were first used on the lower part of Alto Anchicaya Dam, because at that time the dam height was breaking precedents. The detail has since been repeated on Foz do Areia Dam (Pinto *et al.*, 1985); Khao Laem Dam (Watakeekul *et al.*, 1985); Golillas Dam (Amaya and Marulanda, 1985). The designs for recent dams in Colombia (Amaya and Marulanda, 2000) and Brazil (Sobrinho *et al.*, 2000) include upstream protection of the lower section of the face slab.

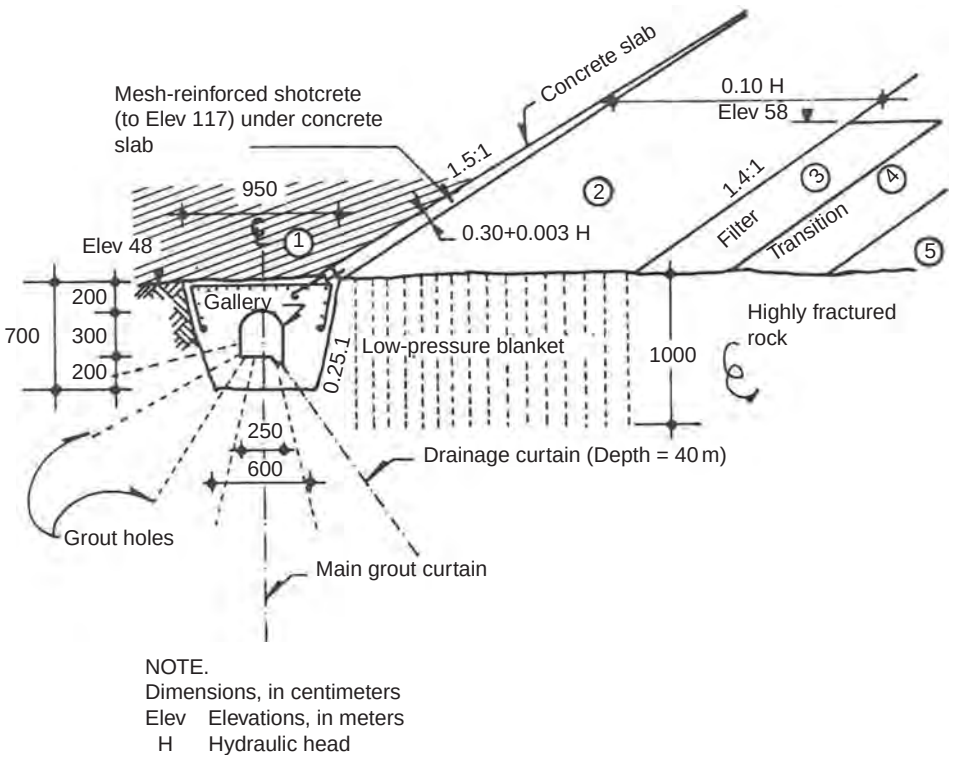


Figure 15.37 Grouting and drainage gallery, Aguamilpa Dam (Moreno, 1987).

The concept is to cover the perimetric joint and plinth in the lower elevations with impervious soil, which would seal any cracks or joint openings. Zone 1A is a minimum practical construction width, with Zone 1B provided for stability.

As pointed out by Cooke and Sherard (1987) many dams have been successfully constructed without this upstream zone, and if Zone 2D is graded fine to act as a filter to dirty fine sand in the event of leakage, there seems little justification for the fill in most dams. They do seem, however, to favour its application in the lower part of the plinth in high dams.

15.5.6 Spillway over the dam crest

Cooke and Sherard (1987) discuss the concept of building the spillway on the downstream face of a CFRD. They suggest that it is practical for ungated moderate size spillways with a peak discharge of around 25–30 m^3/sec per metre of chute width, and where flood flows are of short duration. They suggest the following principles:

- The whole of the rockfill should be compacted to Zone 3A standard to limit settlements.

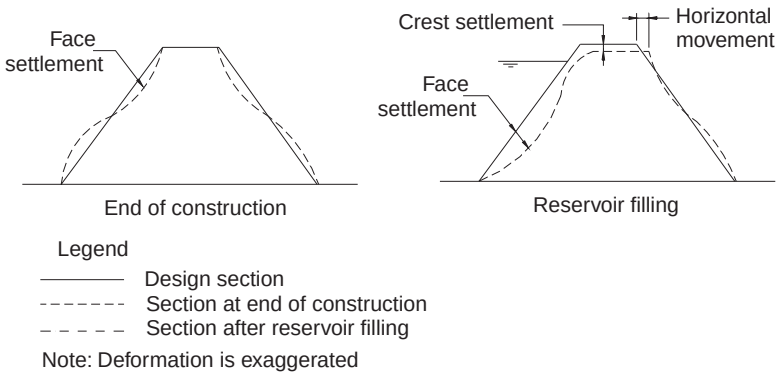


Figure 15.38 Diagrammatic deformation of the CFRD (adapted from Mori, 1999).

- A layer of fine rock should be placed under the concrete, as for the face slab, and this should be rolled to make it a good even and stiff support for the concrete slab in the spillway.
- The spillway chute slab should be built from the bottom up, with continuous reinforcing in both directions, and extend into the side wall footings.
- For high dams, air grooves should be provided in the chute slab. These would also act as contraction joints. Contraction joints would be needed in the spillway walls.

The Hydro-Electric Commission has constructed such a spillway on the 80 m high Crotty Dam. The 12 m wide crest and chute have a discharge capacity of 210 m^3/sec. The chute has four 'hinges' in it to accommodate settlement, aeration steps to prevent cavitation and a flip bucket dissipator.

Carter *et al.* (2000) report that the Crotty Dam spillway has operated a few times since 1991 and has performed very adequately.

15.6 OBSERVED SETTLEMENTS, AND DISPLACEMENTS OF THE FACE SLAB, AND JOINTS

15.6.1 General behaviour

The upstream face of the dam is displaced during construction, on first filling, and during operation (Figure 15.38).

In most cases the face slab is constructed after the fill is all placed, so is not subject to the movements shown to the end of construction.

During first filling, the water load acting on the face slab causes displacement of the face slab normal to the plane of the slab, leading to tension around the perimeter, and compression over the central part of the face, but also causes shear and tensile movements at the perimetric joint. These movements continue with time. They may be sufficient to open joints or cause cracking leading to leakage (Figures 15.39 and 15.40).

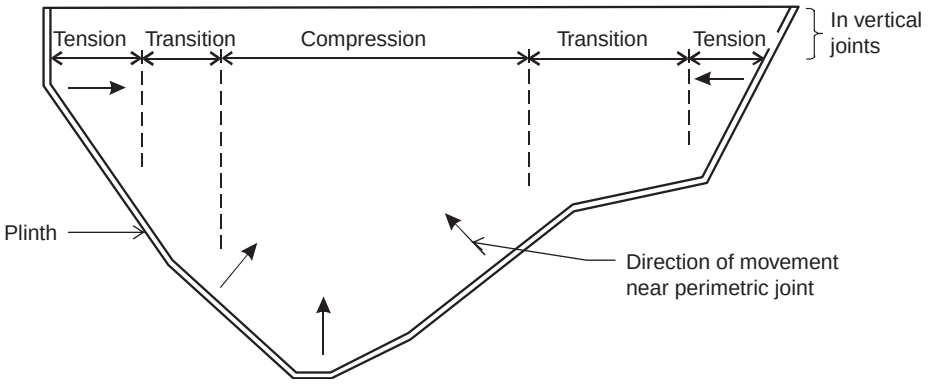


Figure 15.39 Face slab elevation showing movement directions and stress zones.

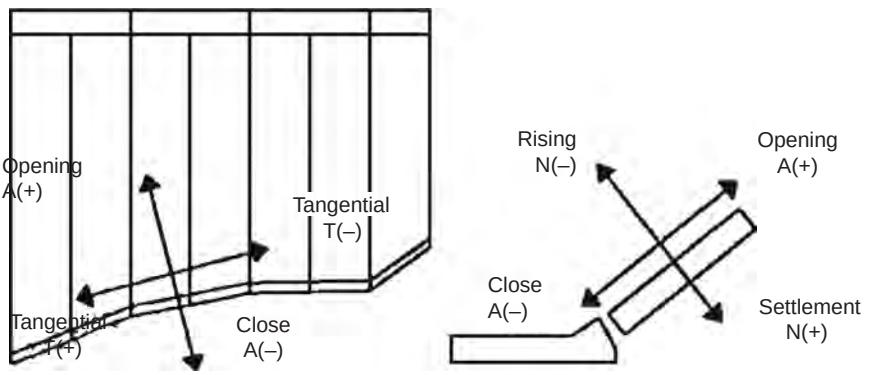


Figure 15.40 Direction of movements at the perimetric joint (Gomez, 1999).

The following data and that in Section 15.7, have been collected to give readers an idea of the range of settlements, displacements and leakage which has been observed, so they can know what to expect, or can gauge the performance of their dam in comparison with others. The data has been prepared with the assistance of Hunter (2003), Ang (2001) and published data. More details on some aspects are available in Hunter (2003), Hunter and Fell (2002, 2003c).

15.6.2 Post construction crest settlement

Available methods (Sowers *et al.*, 1965, Parkin, 1977, Soydemir and Kjaernsli, 1979, Clements, 1984, Pinto and Filho Marques, 1985, Sherard and Cooke, 1987, Hunter, 2003, Hunter and Fell, 2002) for prediction of post-construction crest settlement or rockfill embankments are empirical and are generally based on historical records of similar embankment types and similar methods of construction.

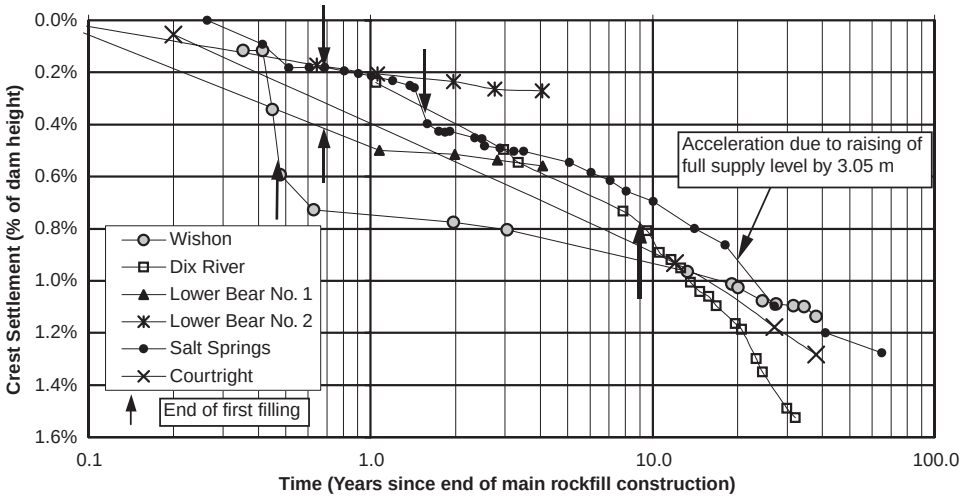


Figure 15.41 Post-construction crest settlement versus time for dumped sluiced rockfill CFRDs (Hunter, 2003).

An important aspect of the deformation behaviour of rockfill related to its stress-strain characteristics is that relatively large deformations occur on application of stresses above those not previously experienced by the rockfill (such as on first filling or embankment raising). On un-loading or re-loading to stress states less than those previously experienced (such as due to fluctuations in the reservoir level) the rockfill modulus is very high and resultant deformation limited. Collapse deformation on initial wetting can result in relatively large deformations. Considerations for the post-construction deformation of rockfill in CFRDs therefore include:

- Events where stresses are likely to exceed those previously experienced; most notably first filling.
- Ongoing, time-dependent (or creep) deformation of rockfill.
- Collapse type deformations due to wetting from leakage or tail-water impoundment.

A significant factor in using empirical methods for prediction is assessment of the base time of deformation and consideration of the timing of events such as first filling. If near full impoundment occurs before completion of construction (or the base time of initial readings) then a significant component of the deformation is likely to have occurred and will not be captured by the post-construction monitoring. For Figure 15.41 to Figure 15.43 and Table 15.10 the initial time has been established at the end of construction of the main rockfill.

The general trend of the vertical crest settlement versus log time plot is for rapid rates of crest settlement during first filling. The steep portions of the curves in Figures 15.41 to 15.43 is typical for CFRDs where first filling started more than about 0.5 years after the end of the main rockfill construction. The steeper portion of the curve is to be expected given that stress levels in the rockfill exceed those previously experienced. This

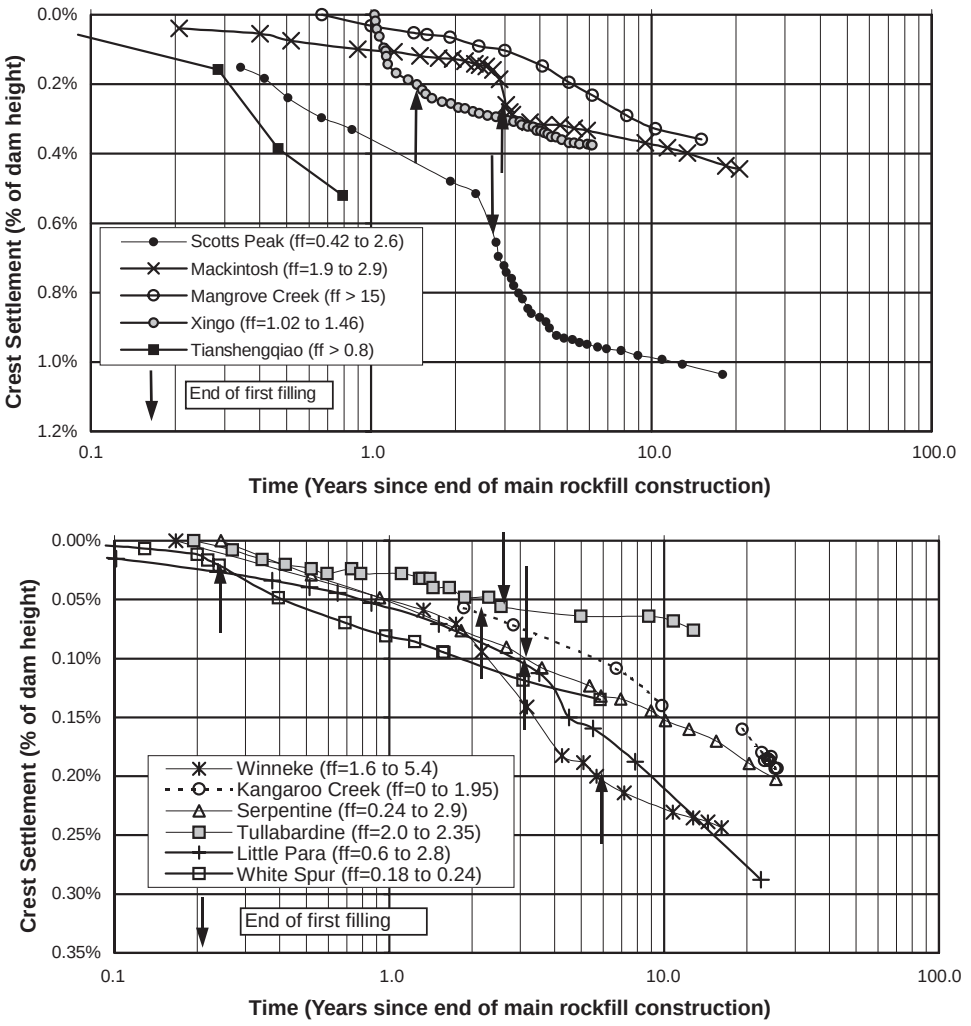


Figure 15.42 Post-construction crest settlement versus time for CFRDs constructed of well-compacted quarried rockfill of medium to high intact strength (Hunter, 2003).

is followed by long-term movements at a near constant creep strain rate (on log scale). The amount of deformation that occurs during first filling comprises a significant proportion of the total crest settlement, greater than 50% in a number of cases.

For the dumped and sluiced rockfill Figure 15.41 indicates:

– Significantly greater magnitude of settlement for the dumped and sluiced rockfill compared with compacted rockfill. The rates of settlement (on log scale) post first filling (time-dependent deformation) are also significantly greater for the dumped rockfill.

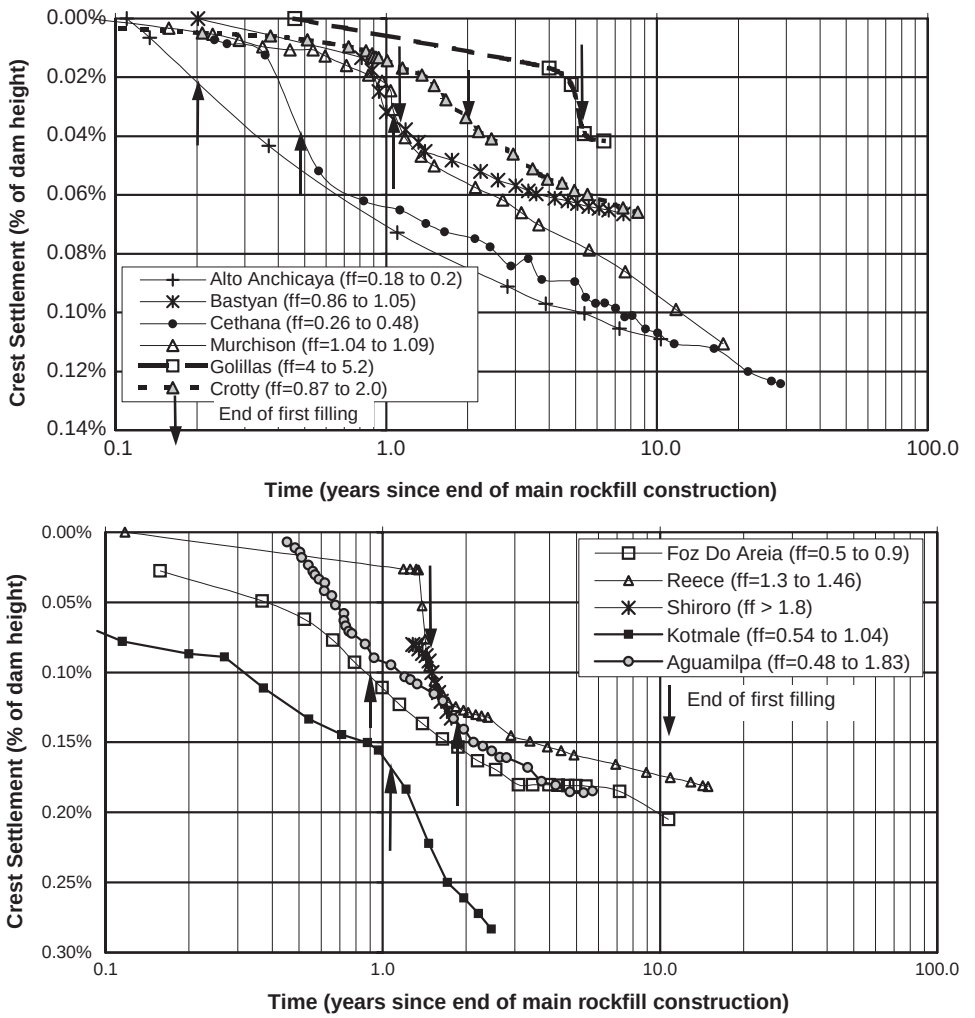


Figure 15.43 Post-construction crest settlement versus time for CFRDs constructed of well-compacted quarried rockfill of very high intact strength and of well-compacted gravels (Hunter 2003).

– The poorly sluiced rockfill (Salt Springs and Dix River) have higher long-term creep rates (strain per log cycle of time) than the well-sluiced rockfill.

For well-compacted rockfill the post-construction crest deformation records presented in Figure 15.42 and Figure 15.43 indicate:

– For embankments constructed of rockfill of medium to high intact strength rock sources, the total magnitude of settlement at 10 years is on average approximately twice that of quarried, very high strength rockfill. The long-term creep rate is also

significantly higher for the weaker rockfill, 2 to 10 times that of quarried, very high strength rockfill.

– For the well-compacted sandy gravels (Crotty and Golillas) the post-construction deformation is less than that for the well-compacted, very high strength quarried rockfill. This is likely to be due to several reasons, but is considered to be mainly a result of the rounded shape of the gravels. The point area of contact between particles of rounded gravel will be significantly greater than that of angular quarried rockfill. Hence, the contact stresses will be significantly less, resulting in less particle breakage.

Table 15.10 summarises the range of total post construction settlement at 10 and 30 years, and the long-term creep rate determined from the data set of CFRDs.

15.6.3 Face slab displacements and cracking

Table 15.11 summarizes maximum observed face slab displacements.

The maximum displacements usually occur at about mid height of the dam, but this may vary depending on the zoning of the rockfill, and whether the face slab was constructed in one stage after all the rockfill was placed, or in several stages. Displacements at the crest are usually less than the maximum but the ratio varies, particularly if the reservoir impoundment begins before the dam was completed (as in Tianshengqiao l dam, Wu *et al.*, 2000).

Table 15.12 summarizes perimetric joint details and maximum observed displacements.

It is apparent from the data presented in Table 15.12 that the maximum opening, settlement and shear measured on perimetric joints of CFRD is typically less than 30 mm.

The larger movements which have been recorded are most likely related to poor compaction and/or use of low modulus, or thick rockfill under the face slab near the perimetric joint.

Large displacements were measured on the perimetric joint of Golillas Dam, predominantly as joint opening. Despite the use of a triple waterstop, large leakage rates were measured during first filling and continue to be measured. There were no apparent low modulus rockfill zones around the perimetric joint and comparatively low strains were measured in the rockfill during and following first filling, as illustrated in Table 15.9. It is likely that the large perimetric joint displacements measured at Golillas are related to the face slab geometry. The dam is located in a steep-sided narrow valley with a crest length to height ratio of 0.81. Similarly large opening and settlement of the perimetric joint were observed at Alto Anchicaya Dam which was constructed in a steep-sided valley. Amaya and Marulanda (2000) observe that "*in both dams the plinth is essentially a vertical wall anchored to the rock, with the rock surface under the plinth having the same inclination.*"

Both Foz Do Areia and Xingo dams have exhibited similar magnitudes of displacement on perimetric joints. They have very similar upslope and down slope face angles and approximately the same crest length to height ratios. Reported face slab movements (Table 15.11) for both these cases were well in excess of typically measured

Table 15.9 Observed post construction crest settlement and horizontal displacements for modern CFRD.

		Horizontal Crest Movement		*Crest Settlement*				
					Total Post-Construction		*Long-Term Settlement Rate*	
Dam Name	*Height (metres)*	*First Filling (mm)*	*Total Post-Construction (mm) (Time from EOC)*	*First Filling (mm)*	*mm (Time from EOC)*	*Vertical Strain (%)*	*% per log time cycle(2,3)*	*mm/year (4)*
Aguamilpa	185.5	n/a	n/a	222	307 (0.4–5.7 yrs)	0.165	0.090	n/a
Alto Anchicaya	140	n/a	n/a	100	153 (0.1–10.3 yrs)	0.109	0.037	At 10 years, settlement rate was less than 2 mm/yr
Bastyan	75	9	16 (0–7.5 yrs)	15	50 (0.2–7.5 yrs)	0.067	0.028	n/a
Brogo	43	n/a	35.7 (0–18.25 yrs)	15	34 (0–18.25 yrs)	0.079	n/a	n/a
Cethana	110	27	83 (0–22 yrs)	46	137 (0–28.6 yrs)	0.125	0.042	n/a
Courtright	97	n/a	n/a	899	1237 (0–38 yrs)	1.275	0.70	n/a
Crotty	83	9	n/a	16	55 (0–8.5 yrs)	0.066	0.045	n/a
Dix River	84	n/a	970 (0–32 yrs)	n/a	1281 (0–32 yrs)	1.525	0.58 to 1.44	At 30 years, settlement rate was approximately 20–25 mm/yr
Foz Do Areia	160	180	248 (0–6 yrs)	73	328 (0–11 yrs)	0.205	0.099	n/a
Golillas	125	n/a	7 (0–6 yrs)	20	52 (0.5–6.4 yrs)	0.042	0.033	n/a
Kangaroo Creek	60	32	50 (0–10 yrs)	26	116 (0–26 yrs)	0.193	0.126	n/a
Kotmale	90	n/a	62 (0–2.5 yrs)	96	255 (0–2.5 yrs)	0.283	0.258	n/a
Little Para	53	N/a	n/a	22	152 (0–22.6 yrs)	0.287	0.210	n/a
Lower Bear No. 1	75	270	305 (0–4 yrs)	<335	375 (0.1–4.1 yrs)	0.500	0.103	n/a
Lower Bear No. 2	46	88	116 (0–4 yrs)	73	116 (0.1–4.1 yrs)	0.252	0.128	n/a
Mackintosh	75	75	130	99	333 (0–20.6 yrs)	0.444	0.184	At 10 years, settlement rate was 5 mm/yr
Mangrove Creek	80	n/a	196 (0–15 yrs)	>287	287 (0.7–15 yrs)	0.359	0.251	n/a
Murchison	94	8	22 (0–12 yrs)	9	104 (0.1–17.6 yrs)	0.111	0.056	At 10 years, settlement rate was approximately 3 mm/yr

Table 15.9 Continued.

		Horizontal Crest Movement		*Crest Settlement*				
					Total Post-Construction		*Long-Term Settlement Rate*	
Dam Name	*Height (metres)*	*First Filling (mm)*	*Total Post-Construction (mm) (Time from EOC)*	*First Filling (mm)*	*mm (Time from EOC)*	*Vertical Strain (%)*	*% per log time cycle(2,3)*	*mm/year (4)*
Reece	122	n/a	68 (1.5–9 yrs)	85	221 (0.1–15 yrs)	0.181	0.063	At 5 years, settlement rate was approximately 4 mm/yr
Salt Springs	100	230	550 (0–27 yrs)	380	1276 (0.3–65 yrs)	1.276	0.29 to 0.77	n/a
Salvajina	148	n/a	n/a	>90	90 (0.3–0.8 yrs)	0.061	n/a	n/a
Scotts Peak	43	96	190 (0–17 yrs)	203	445 (0–18 yrs)	1.035	0.178	Long term rate approx 2.5 mm/yr
Segredo	145	n/a	n/a	Approx 200	229 (0–0.8 yrs)	0.158	n/a	n/a
Serpentine	38	23	38 (0–21 yrs)	35	77 (0.2–25.5 yrs)	0.203	0.109	n/a
Shiroro	125	n/a	27 (0–1.5 yrs)	>66	166 (0–1.8 yrs)	0.133	n/a	n/a
Tianshengqiao–1	178	670 (0-0.75 yrs)		>926	926 (0.1–0.8 yrs)	0.520	n/a	n/a
Tullabardine	25	n/a	n/a	2	19 (0.2–12.8 yrs)	0.076	0.023	n/a
White Spur	43	n/a	n/a	7	58 (0–5.9 yrs)	0.135	0.080	At 5 years, settlement rate was approx 1.25 mm/yr
Winneke	85	n/a	n/a	105	207 (0.2–16.2 yrs)	0.244	0.107	n/a
Wishon	90	n/a	n/a	189	954 (0–38 yrs)	1.060	0.25 to 0.33	n/a
Xingo	140	210	320 (1–6.2 yrs)	302	526 (1.0–6.2 yrs)	0.376	0.250	n/a

Notes:
1. EOC = end of construction of main rockfill
2. Time in years
3. Effects of first filling typically cease 6 months after completion of first filling
Quoted time is from end of construction

Table 15.10 Post construction total crest settlement and long-term creep rate for CFRDs (Hunter, 2003, Hunter and Fell, 2002).

Rockfill Classification	*Total Post Construction Settlement (% of dam height)*		*Long-term Creep Rate (%/log cycle)*
	10 years	*30 years*	
Dumped Rockfill	0.6 to 1.0%	1.0 to 1.5%	0.3 to 1.5
Well Compacted Rockfill:			
Medium to high strength	0.15 to 0.4	–	0.05 to 0.25
Very high strength, quarried	0.06 to 0.2	–	0.02 to 0.10
Gravel Rockfill	0.2 to <0.05	<0.10	

Note: (1) Rock substance unconfined compressive strength medium 6–20 MPa, high 20–70 MPa, very high 70–240 MPa.

deformations on perimetric joints and face slabs, which is likely to be related to the nature of zoning within the rockfill and variations in modulus between zones. Both dams had upstream zones of well compacted angular rockfill and large downstream zones of moderately compacted rockfill. Hunter and Fell (2002, 2003c) reported large variations in average secant modulus during construction between these two zones in both dams. Perimetric joint movement at Xingo may also have been related to irregular left abutment and foundation geometry. Variations in rockfill thickness between the face slab and foundation resulted in differential settlements behind the face slab and eventually cracking in the face slab.

15.6.4 Cracks in CFRD dams

The progressive development of CFRD embankments has resulted in improved design of the rockfill, plinth and face slab with associated reduction in leakage. However as higher and higher dams are built design peculiarities have been associated with leakage incidents. These incidents appear to have resulted from either foundation features or differential deformation of the rockfill (Marulanda and Pinto, 2000).

Mori (1999) identified three general types of face slab cracking:

i) Type A – shrinking cracking associated with concrete curing. These cracks are typically horizontal, very small width (<1 mm) and limited in extent to the width of a slab.
ii) Type B – associated with bulging of the lower part of the rockfill embankment. Mori observed that, in cases where upstream earthfill is in place; this type of cracking is usually restricted to the middle third of the dam height.
iii) Type C – cracks associated with settlement of the rockfill embankment, particularly differential settlements related to staged construction or zoning of materials with contrasting moduli.

Early CFRD dams, such as Courtright, Salt Springs and Wishon, suffered substantial face slab cracking prior to and during first filling. This damage is attributable

Table 15.11 Maximum observed face slab displacements for modern concrete face rockfill dams.

		Total Displacement Normal to Face Slab (1)						
			Long-Term Displacement (2)			*Displacement Rate (mm/year)*		
Dam Name	*Height (metres)*	*Displacement on First Filling mm*	*mm*	*Depth of Maximum Movement (below crest)*	*Comment (3)*	*First Filling (time since start of first filling)*	*Long Term (time since end of first filling)*	*Comments*
Aguamilpa	185.5	n/a	320	0 m	For 6 years from EOC	n/a	n/a	n/a
Alto Anchicaya	140	130	160	n/a	For 3 years from EOC	n/a	n/a	n/a
Bastyan	75	56	68	34 m	For 7.5 years from EOC	155 (0.33 yrs)	4 (0.2–2.5 yrs) 0.5 (2.5–6.5 yrs)	Filling over 0.2 yrs
Cethana	110	114	170	37 m	For 22 years from EOC	145 (0.83 yrs)	6 (0.6–3.9 yrs) 2.8 (3.9–11 yrs) 0.9 (11–22.4 yrs)	Filling over 0.23 yrs
Chengbing	74.6	154	185	58 m	For 10 years from EOC	51 (3 yrs)	1.9 (0–3.7 yrs)	Filling over approx. 5 yrs
Crotty	83	35	46	49 m	For 2.5 years from EOC	75 (0.47 yrs)	10.2 (0.15–1.25 yrs)	Filling over 0.3 yrs
Foz Do Areia	160	620	780	82 m	Duration unknown	1500 (0.42 yrs)	33 (2–3 yrs)	Filling over 0.42 yrs
Golillas	125	160	n/a	68 m	Recorded up to 0.5 years following completion of first filling	n/a	n/a	n/a
Guanmenshan	58.5	n/a	25.6	29 m	For 5 years from EOC	n/a	n/a	n/a
Ita	125	461	n/a	n/a	n/a	n/a	n/a	n/a
Kotmale	90	98	–	–	–	–	–	–
Longxi	59	–	19.1	30 m	For 2.75 years from EOC	–	–	–
Lower Bear No. 1	75	550	625	42 m	For 2 years from EOC	600 (0.9 yrs)	115 (0.4–0.6 yrs) 13 (0.6–3.4 yrs)	Filling over 0.5 yrs
Lower Bear No. 2	46	137	165	15 m	For 4 years from EOC	120 (1 yr)	11 (0.4–3.4 yrs) 0.5 yrs	Filling over

Murchison	94	28	77	67 m	For 12 years after EOC	130 (0.2 yrs)	17 (0–0.4 yrs) 5 (0.4–2.5 yrs) 1.4 (2.5–10.5 yrs)	Filling over 0.05 yrs
Reece	122	215	264	66m	For 10 years after EOC	860 (0.25 yrs)	30 (0.1–0.4 yrs) 12 (0.4–2.4 yrs) 3.5 (2.4–5.4 yrs) 3.5 (5.4–8.4 yrs)	Filling over 0.15 yrs
Salt Springs	100	1317	1755	67m	For 28 years after EOC	610 (2 yrs)	38 (0.5–2.5 yrs) 18 (2.5–8.5 yrs) 12.5 (8.5–25.5 yrs)	Filling over 1.5 yrs
Salvajina	148	55	–	–	Measured during first filling up to 14 m below FSL	–	–	–
Scott's Peak	43	78	178	–	For 18 years after EOC	–	–	–
Segredo	145	340	n/a	n/a	Measured up to 4 months after completion of first filling	n/a	n/a	n/a
Shiroro	125	90	n/a	n/a	Measured during first filling up to 32 m below FSL	n/a	n/a	n/a
White Spur	43	15	38	12 m	For 6 years after EOC	75 (0.2 yrs)	7.5 (0–0.7 yrs) 23 (0.7–1.1 yrs) 2.5 (1.1–5.5 yrs)	Filling over 0.06 yrs
Winneke	85	145	160	n/a	For 1.5–15.5 years after EOC	n/a	n/a	n/a
Xibeikou	95	75	n/a	n/a	n/a	n/a	n/a	n/a
Xingo	140	290	510	n/a	For 6 years from EOC	n/a	n/a	n/a

Notes:
(1) Total displacement includes displacement measured during first filling
(2) EOC = end of construction of main rockfill FSL = Full supply level
(3) Displacements are measured normal to the face slab.

Table 15.12 Perimetric joint details and maximum observed displacements for modern concrete face rockfill dams.

	Water Stop Details			*Maximum Displacement (mm)*				
Dam Name	*Bottom*	*Middle*	*Top*	*Opening*	*Settlement*	*Shear*	*Comments – Timing*[1]	*Comments – Location*
Aguamilpa	Copper	PVC	Flyash	25	18	5.5	Opening and settlement measured to 7 yrs after EOC. Shear measured at 2 yrs after EOC.	Maximum opening & settlement measured towards right abutment. Max shear measured towards centre of dam.
Alto Anchicaya	None	Rubber	None	125	106	15	Measurements at less than 8 yrs after EOC.	Remediation included application of compacted clay, sand-asphalt mix & mastic to perimetral joint.
Bastyan	Stainless steel	Rubber	None	4.8	21.5	Unknown	Measured to 7.5 yrs after EOC.	Details of water stop not available
Cethana	Copper	Rubber	None	12	Unknown	7.5	Measured approx. 12 yrs after EOC.	Maximum opening in centre of dam. Maximum shear in left abutment.
Chengbing	Copper	PVC	Mastic	13.1	28.2	20.6	Measurements at 1–2 yrs after EOC.	Maximum movement on lower part of left abutment.
Crotty	Stainless steel	Hypalon	None	2	3.3	24	All readings taken approx. 2 yrs after first filling.	Max shear on left abutment. Max opening and settlement close to centre of dam.
Foz Do Areia	Copper	PVC	Mastic	23	55	25	Timing unknown.	Location of maximum movements unknown.
Golillas	Copper	PVC	Mastic	100	36	Unknown	Measurements at less than 11 yrs after EOC.	Location of maximum movements unknown.
Guanmenshan	Copper	Rubber	Rubber asphalt	5.7	4.8	2.8	Measurements within 5 yrs of EOC.	Location of maximum movements unknown.
Ita	Copper	None	Mastic	Unknown	Unknown	Unknown	No data available.	No data available.
Khao Laem	Copper	Hypalon	Mastic	5	8	22	Measurements within 16 yrs of EOC.	Location of maximum movements unknown.

Kotmale	Unknown	Unknown	Unknown	2	20	5	No data on nature of water stops available.	Location of maximum movements unknown.
Machadinho	Copper	None	Mastic	Unknown	Unknown	Unknown	No data available.	No data available.
Mackintosh	Stainless steel	Stainless steel	None	6.5	22	2.5	Maximum opening & shear measured 7 yrs after EOC. Max settlement at 6 yrs after EOC.	Max settlement on right abutment.
Mangrove Creek	Copper	Rubber	None	12	57	6	Measurements to approx. 11 yrs after EOC.	Max settlement on right abutment and max shear on left abutment. Max opening in centre of dam.
Murchison	Stainless steel	Rubber	None	20	10	16	Initial max settlement approx 10 mm measured during first filling. 4 mm rebound measured in 10 yrs following filling.	Max shear and opening on left abutment. Max measured settlement at centre of dam.
Outardes 2	Copper	PVC	None	3	Unknown	Unknown	Max measured opening of 3 mm measured during first filling.	No data on settlement or shear.
Paloona	Copper	Rubber	None	0.5	5.5	Unknown	Measurements to approx. 7 yrs after EOC.	Location of maximum movements unknown.
Reece	Stainless steel	Hypalon	None	10	27	Unknown	Measurements to 5 yrs after EOC.	Maximum movements on perimetric joint close to centre of dam.
Salvajina	Copper	PVC	Mastic	9	19.5	6	Measurements at less than 15 yrs after EOC.	Location of maximum movements unknown.

(*Continued*)

Table 15.12 Continued.

	Water Stop Details			*Maximum Displacement (mm)*				
Dam Name	*Bottom*	*Middle*	*Top*	*Opening*	*Settlement*	*Shear*	*Comments – Timing* [1]	*Comments – Location*
Santa Juana	Copper	None	Mastic	Unknown	Unknown	Unknown	No data available.	No data available.
Segredo	Copper	None	Mastic	Unknown	Unknown	Unknown	No data available.	No data available.
Serpentine	None	Rubber	None	3	6.5	Unknown	Max settlement 5 yrs after EOC. Max opening 3.5 yrs after EOC.	No data on location of maximum movement.
Shiroro	PVC	Rubber	None	30	58	21	Measurements to 2 yrs after EOC.	Max opening in left abutment. Max shear & settlement in right abutment.
Tianshengqiao-1	Copper	PVC	Mastic	16	22.5	7	Measurements within 6 months of EOC.	Max opening in centre of dam, max settlement in left abutment and max shear on right abutment.
Tullabardine	Stainless steel	None	None	1.5	8.5	0.3	Measurements up to 5 yrs after EOC.	Max opening measured on left abutment. Max settlement measured on right abutment.
White Spur	Stainless steel	None	None	3	8	Unknown	Max settlement to 5.5 yrs after EOC and max opening measured at 1.5 yrs after EOC.	Measurements only made at centre of dam.
Winneke	Copper	Rubber	None	9	19	24	Timing unknown.	Location of maximum movements unknown.
Xibeikou	Copper	PVC	Mastic	14	24.5	5	Measurements to 10 yrs after EOC.	Max shear, opening and settlement measured on left abutment.
Xingo	Copper	None	Mastic	30	29	46	Measurements within 6 yrs of EOC.	Location of maximum movements unknown.

Note:
1. EOC = end of construction of main rockfill

to the large settlements (typically in excess of 1% vertical strain) that occurred in the dumped and sluiced rockfill embankments.

Regan (1997) describes remedial works used in some of these early dams, to reduce leakage rates through the face slab.

Changes in construction methods in about the 1960s, from these early dumped and sluiced embankments to well zoned and compacted embankments, have brought about substantial reductions in embankment deformations. Marulanda and Pinto (2000) report that "monitoring of the performance of compacted CFRD dams has indicated that most of the face is under biaxial compression", with small tensile stresses observed near the toe and crest of the dam and around the perimeter. However, recent performance of face slabs on some new CFRD dams may suggest that tensile stresses can develop within the face slab under particular conditions.

At Khao Laem in Thailand (130 m high) vertical cracking of the face slab occurred above a foundation ridge. In this dam, apart from a few contraction joints the face slab is a continuously reinforced membrane with steel extending across the vertical construction joints. The cracks appear to have resulted from differential embankment deformation across the foundation ridge. Watakeekul *et al.* (1985) note that cracking may have occurred due to an inadequate number of contraction joints within the face slab. The authors understand that after some attempts to repair these cracks with grouting, J.B. Cooke suggested sand be dropped in the crack, which resulted in a rapid drop in seepage.

At Aquamilpa Dam in Mexico (187 m high) several cracks have occurred in the upper section of the face slab, as illustrated in Figure 15.44. Marulanda and Pinto (2000) report that a 165 m long crack was found 55 m below the crest, approximately three years after completion of first filling. The cause of this cracking has been attributed to the large difference in modulus between the compacted gravel alluvium in the upstream half of the zoned embankment and the rockfill in the downstream half of the embankment (Gomez *et al.*, 2000). Modulus values for the gravel and rockfill were estimated to be 200–300 MPa and 40–60 MPa respectively (Marulanda and Pinto, 2000). Similarly large differences in modulus between zones were reported for Salvajina (ICOLD, 1989a), yet no cracking has been observed. Marulanda and Pinto (2000) suggest that this difference in behaviour is related to the gradient between zones. The contact between upstream and downstream zones at Salvajina had a slope of 0.5V:1H, while at Aguimilpa the contact between the upstream gravel fill and the transition zone is vertical. Differential settlements in the upper half of the embankment resulted in development of tensile stresses in the face slab and consequent cracking.

Campos Novos dam is a 202 m high dam completed in 2006. It is built in a narrow valley with basalt rockfill which resulted in a rockfill modulus of 50–60 MPa. In 2005 during filling of the reservoir the central compression vertical joint failed as shown in Figure 15.20. This was accompanied by shifting and superposition of the lateral slabs by 12 to 15 cm at about 30% to 40% of the dam height over a length of about 300 m as shown in Figure 15.45. There was distortion of reinforcement, rupture of copper water stops and leakage up to 1500 l/sec. (Cruz *et al.*, 2009; Antunes Sobrinho *et al.*, 2007). Readers are referred to those papers for more details on why the cracking occurred.

Cracking was observed at Scott's Peak dam, a dam with an asphaltic concrete face, again due to the use of rockfill zones with marked contrast in modulus. Well

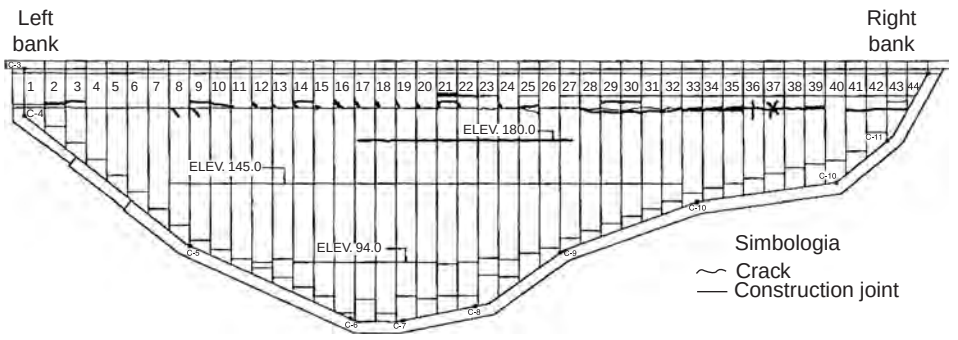


Figure 15.44 Aguamilpa Dam, localization of the cracks in the concrete face.

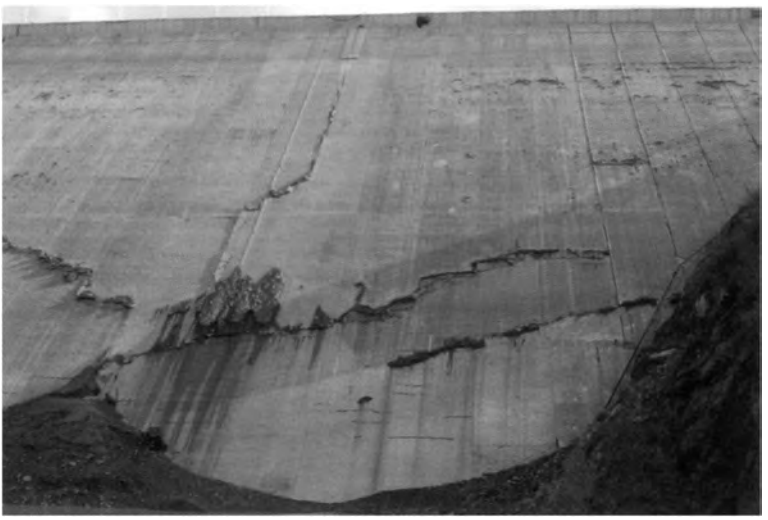


Figure 15.45 Campos Novos Dam face slab failure (Cruz *et al.*, 2009).

compacted gravels were placed at the upstream toe (to a height of approximately one-third the total embankment height), while the main embankment comprised compacted argillite rockfill. On first filling, large differential deflections caused tensile cracking of the upstream face near to the contact between the gravel and argillite rockfill zones, as illustrated in Figure 15.46. There was also a rapid increase in the rate of settlement in the short period before gravels were dumped over the damaged area of the face.

Xingo Dam in Brazil (140 m high) has been built in several stages. Horizontal cracking of the face slab has occurred near the stage interface indicating differential deformation in the embankment between the materials placed in the different stages.

At Guohou in China failure is attributed to a crack that opened at the joint between the face slab and the crest wall following deformation of the zoned gravel embankment (Chen, 1993). However the detailing and construction of this joint was poor.

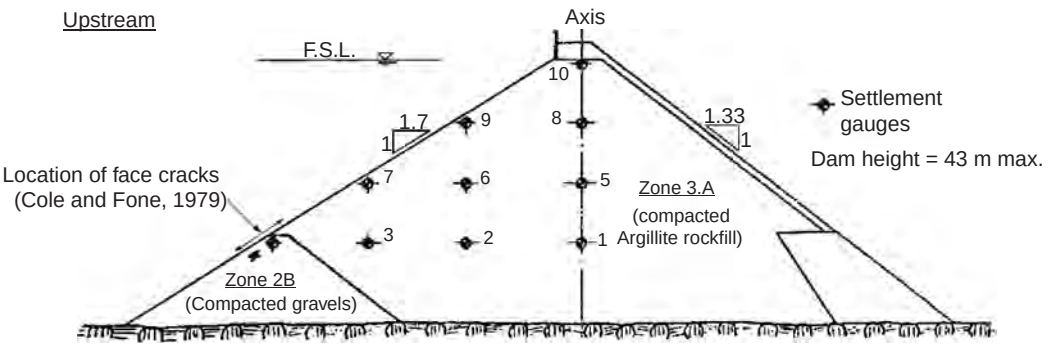


Figure 15.46 Scotts Peak dam (courtesy of HEC Tasmania).

These features indicate that in the case of the higher CFRD embankments it is important to limit the potential for differential deformation of the rockfill, particularly in the higher sections of the dam, to have a modern Zone 2D to limit leakage, and to provide high permeability zones to discharge the leakage safely.

Performance of Zipingpu CFRD under large seismic loading

The 156 m high Zipingpu CFRD dam was impacted by a magnitude 8 Wenchuan earthquake in May 2008. The dam is sited 17 km from the epicentre and recorded peak ground accelerations of over 2g at the crest with a bedrock PGA of 0.5 g. The damage to the dam is described in Wieland (2009), Cruz *et al.* (2009), Chen (2008a, b) and Xu (2008).

The most significant damage was to vertical joints which suffered compressive stress failure and offset to a horizontal joint between the second and third stage face slab. Figures 15.47 and 15.48 show the damage. There was also lateral deformation at the crest as shown in Figure 15.49.

The maximum settlement at the dam crest due to this massive earthquake loading was 735 mm (0.5% of dam height) and 180 mm downstream. The water level was low (54 m below crest level) in the dam at the time of the earthquake. Maximum leakage went from 10 l/sec to 18 l/sec.

This is an important case study because it shows that the face slabs of CFRD are vulnerable to damage under large seismic loads because the settlements induced by the earthquake are far larger than experienced under normal operating conditions (see Table 15.10).

Zipingpu Dam; was a modern design so the Zone 2D would have limited leakage even if the reservoir level had been higher. However many older CFRD do not have a modern Zone 2D to limit leakage flows and have fewer and more vulnerable perimeter and other joints. If they also have other than free draining rockfill they may be vulnerable to failure as the rockfill saturates by large flows in the damaged face slab.

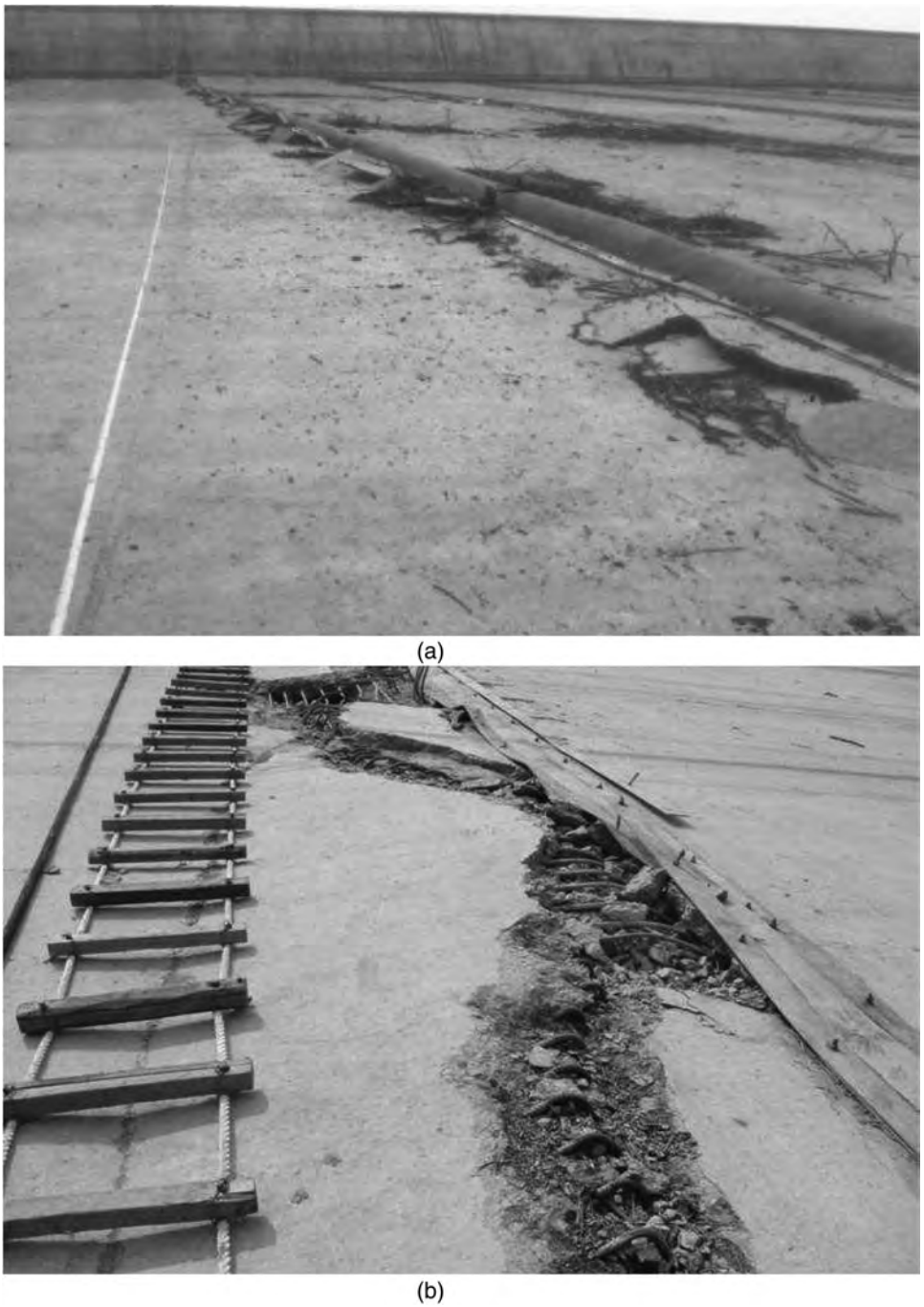


Figure 15.47 Damage of vertical joint of face of Zipingpu CFRD looking upwards towards the crest (a) damage, (b) preparation for repairs, (Wieland, 2009; photographs courtesy of Xu Zeping, China).



Figure 15.48 Damage of horizontal joint between Stages 2 and 3 of the concrete face of Zipingpu CFRD (Wieland, 2009, photographs courtesy Xu Zeping, China).



Figure 15.49 Damage at downstream crest of Zipingpu CFRD (Wieland, 2009, photograph courtesy Xu Zeping, China).

15.7 OBSERVED LEAKAGE OF CFRD

15.7.1 Modern CFRD

Table 15.13 shows leakages which have been recorded on first filling and during operation for a selection of dams for which data is available.

The following case studies demonstrate some important features of dams which have experienced relatively large leakage:

(a) *Alto Anchicaya Dam* – high leakage rates measured during first filling (>1800 l/sec in the right abutment). Flooding from a creek on the right abutment during construction caused some wash-out of filter material beneath the perimetric joint and may have been a contributing factor to the excessive leakage. A single rubber water stop was adopted in the construction of Alto Anchicaya. Inspections of the perimetric joint during remedial treatment revealed that while the rubber water stop was intact it was relatively loose and concrete had not penetrated under the water stop very well during construction (Regalado *et al.*, 1982). Remedial treatment included filling the joints with a mastic compound and a sand-asphalt and/or compacted clay cover. Long-term leakage rates stabilised after remedial works, to a rate of approximately 130 l/s.

(b) *Aguamilpa Dam* – leakage rates measured during first filling were considered relatively normal until the water level reached a depth of approximately 173 metres, at which point the rate rose to approximately 260 l/s. Several horizontal and diagonal cracks were found in the face slab at a water depth of approximately 150 metres (Gomez 1999). It is suggested in Gomez (1999) that these cracks could have developed due to tensile stresses generated as a result of differences in modulus between the compacted alluvium (upstream half of embankment) and rockfill (downstream half of embankment). Remedial works to reduce the leakage rate have included the placement of silty sediments over the cracks. Recent dam performance shows leakage rates respond to reservoir level and rainfall events, with an overall long-term trend (1995–2000) of increasing leakage rate based on data presented by Gomez (1999).

(c) *Golillas Dam* – Amaya and Marulanda (1985) reported significant leakage during first-filling of Golillas Dam with distinct relationship between reservoir level and leakage rate. Maximum reported leakage rate during first filling was in excess of 1000 l/s, primarily associated with the perimetric joint and opening of joints within the rock in the left abutment. Initial remedial works, involving removal of the PVC and mastic water stops in some areas and replacement with burlap, lead fibre and mastic, did not result in any real reduction in leakage. Drawdown of the reservoir enabled cleaning and re sealing of joints in the abutment and abutment support using shotcrete. This remedial work, along with the washing of fines into the perimetric joint, resulted in reduction in leakage rate to long-term values of the order of 270–500 l/s.

15.7.2 Early CFRD and other dams which experienced large leakage

Early designs of CFRD (1920s–1960s) were typically characterized by a main body of dumped rockfill and a thick upstream zone of derrick-placed rockfill supporting

Table 15.13 Leakage measured in modern CFRD.

			First Filling		*During Operation*				
Dam Name	*Height (m)*	*Water Depth at Full Supply Level (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Long-Term Rate (l/sec)*	*Comments on Leakage Trends*	*Comments*
Aguamilpa	185.5	182.5	260	169.5	150–200	165–175	50–100	Seasonal fluctuations in leakage rate related to reservoir level. Peak leakage rate at peak reservoir levels.	Face slab cracking developed on first filling. Long-term rate shows steady increasing trend.
Alto Anchicaya	140	135	>1800	125	450	135	130	Seasonal trends in leakage rate are unknown. Rate known to vary with rainfall and reservoir level.	Remedial works following first filling to fix joints in slab.
Bastyan	75	70	10	70	10–25	70	5	No fluctuations in reservoir level. Leakage fluctuations related to rainfall events. Long-term leakage rate only fluctuates by 2–3 l/s.	Long-term near constant leakage rate established approx. 5 years after first filling.
Brogo	43	28.5	42.5	28.5	5–10	30	4–5	Progressive decrease in leakage rate to 4–5 l/s over 8 years following first filling.	Near constant long-term rate with little apparent relationship to rainfall or reservoir level.
Cethana	110	101	60–70	Approx. 80	20–80	96	5–10	Max recorded leakage related to rainfall events. Long-term base leakage rate stable at 5–10 l/s.	Base rate stabilised 4 years after first filling.
Chengbing	74.6	69.5	75	Unknown	60	63.5	20	Fluctuations in leakage rate closely related to reservoir level. Min rate 10 l/s at water depth less than 50 m.	Max leakage during construction when reservoir level rose above face slab

Table 15.13 Continued.

			First Filling		*During Operation*				
Dam Name	*Height (m)*	*Water Depth at Full Supply Level (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Long-Term Rate (l/sec)*	*Comments on Leakage Trends*	*Comments*
Crotty	83	75	45–50	67	50–70	70–75	30–35	Max recorded leakage related to rainfall events. Long-term base leakage rate reduced from 45–50 l/s to 30–35 l/s over 2 years.	Base rate stabilised 2 years after first filling.
Foz Do Areia	160	155	236	150.5	Unknown	Unknown	60	Long-term rate at water depth of approx. 150 m. Fluctuations in leakage rate unknown.	Long-term rate established 5 years after first filling.
Golillas	125	122.5	1080	115	600–700	115	270–500	Fluctuations in leakage rate are unknown.	Long-term rate established approx. 15 years after completion of first filling
Guanmenshan	58.5	57	10	57	Unknown	Unknown	5–6	Little change in leakage rate in 4–5 yrs since first filling.	No apparent relationship between reservoir level and leakage rate.
Ita	125	119.5	Unknown	Unknown	1700	Unknown	380	Long-term rate established by dumping sand/silty sand on face slab cracks.	Max leakage rate measured 4 months after first filling due to face slab cracking.
Kangaroo Creek	60	48	Unknown	Unknown	11	Unknown	2.5	Long-term rate occurs at water depths greater than 25 m. Less than 25 mm and leakage rate appears to a reduce.	Peak rates related to rainfall events.
Kotmale	90	85.6	10–40	40–80	30–50	85	10–20	Seasonal fluctuations in leakage rate due to reservoir fluctuations. Minimum rate of 10 l/s at water depth of 40–45 m.	Rainfall events also cause peak in leakage rate.

Little Para	53	50.5	16	43	18	50.5	3–5	Seasonal fluctuations in leakage rate related to reservoir level. Peak leakage rate at peak reservoir levels.	Max flow measured just prior to overflow event
Mackintosh	75	73	25	62	20–25	68	8–10	Fluctuations in leakage rate related to 5–10 m fluctuations in reservoir level.	Gradual decline in leakage rate over 10 years since first filling.
Mangrove Creek	80	74	Unknown	Unknown	5.6	67.5	2.5	Seasonal fluctuations in leakage are unknown. Increase in leakage rate observed with higher water depth due to face slab cracking.	Long-term rate of 2.5 l/s established at water depth of 46 m, 15 years after commencement of first filling.
Murchison	94	85	5–10	70	10	73	2	Short duration peak leakage up to 15–25 l/s related to rainfall events. Increase in rate related to reservoir level (10 l/s at FSL, <5 l/s at <75 m water depth).	Long-term base rate decreased to 2 l/s 5–6 years after first filling. Steady increase to 5–10 l/s 10 years after first filling.
Salvajina	148	141	Unknown	Unknown	74	130	Unknown	Seasonal trends in leakage rate are unknown.	60 l/s through slab, 14 l/s through abutments.
Scotts Peak (asphaltic concrete face)	43	39.5	100	37	6–10	39	2–3	Short duration peak leakage up to 6–10 l/s likely related to rainfall events. Reservoir held at near constant level.	Max rate occurred after cracking in slab. Long-term rate of 5 l/s for 4 years after first filling then gradual reduction to 2–3 l/s over the next 10 years.
Segredo	145	141	390	141	Unknown	Unknown	45	Seasonal changes in leakage rate are unknown.	Rate of 45 l/s established approx. 6 months after completion of first filling at water depth of 138 m.

Table 15.13 Continued.

			First Filling		*During Operation*				
Dam Name	*Height (m)*	*Water Depth at Full Supply Level (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Long-Term Rate (l/sec)*	*Comments on Leakage Trends*	*Comments*
Shiroro	125	112.5	1600	105	Unknown	Unknown	100	Cracking in face slab occurred during first filling. Seasonal trend in leakage rate is unknown.	Sand placed on face slab & leakage reduced to 100 l/s 6 months after commencement of filling.
Tianshengqiao – 1	178	167	132	154.5	N/A	N/A	N/A	Face slab cracking occurred during first filling.	First filling data not complete.
Tullabardine	25	23	Unknown	Unknown	3–6	22–23	1–2	Minimum leakage rate at lowest water levels. Less than 1 l/s at water levels less than 14 m. Seasonal fluctuations in reservoir level reflected in leakage rates.	Seasonal fluctuations in leakage rate with progressive long term decrease in average rate to 1–2 l/s approx. 10 years after first filling.
White Spur	43	41	7	38	12	30	2	Short duration fluctuations in leakage rate likely related to rainfall events. Reservoir level near constant.	Long-term rate of 2 l/s at water depth of 34 m established approx. 3 years after first filling.
Winneke	85	82	60	81	25–30	Unknown	15	Progressive increase in leakage rate with water depth during first filling. Approx 20 l/s at water depth of 40 m and 50 l/s at water depth of 60 m.	General long term trend of reduction in leakage rate. Long-term rate of 25 l/s 6 years after first filling reduced to 15 l/s 10 years after first filling.
Xingo	140	137	160	133	200	137	140	Initial decrease in leakage rate for one year after completion of first filling. Then increase from 100 l/s to 200l/s over two years due to cracking in face slab. Reservoir level maintained at full supply level.	Max rate due to cracking in slab. Steady decrease to near constant 140 l/s approx 5 years after first filling.

the concrete face. Over the period from 1920 to 1960 embankment construction involved dumping in high lifts (up to 20–50 m) followed by sluicing. The minimal control on placement of the rockfill resulted in large deformations within the embankment as illustrated in Table 15.8, for dams such as Dix River, Salt Springs, Courtright and Wishon. Typical post-construction vertical strain for these cases was in excess of 1% (compared to modern CFRD cases which typically show less than 0.2% vertical strain).

As a result of these large settlements, many early CFRD developed significant face cracking and opening of joints (particularly perimetric joints), resulting in high leakage rates. Table 15.14 presents data on leakage rates observed in early CFRD during first-filling and during operating.

Other dams which have experienced large leakage or flows through rockfill include Scofield Dam (Sherard, 1953) which experienced leakage on a piping incident of 1400–5600 l/sec, and Hell Hole, which began unravelling in a flood during construction at a flow of 540 m^3/sec (Leps, 1973).

15.8 FRAMEWORK FOR ASSESSING THE LIKELIHOOD OF FAILURE OF CFRD

15.8.1 Overview of approach

CFRD are inherently very unlikely to fail, provided they are designed and constructed with adequate permeable zones (e.g. free draining rockfill, or screened and washed gravels in 'gravel' dams) to accommodate large leakage flows and have a modern Zone 2D zone to limit leakage flows. The concrete of the face slab, and surrounding the perimetric and other joints or cracks, which may form in the face slab, is essentially non erodible, so leakage should not develop rapidly so for well managed dams intervention to limit leakage flows should be possible.

However in existing dams the rockfill, and even the Zone 2D and 2E materials are often poorly graded, and potentially subject to internal instability, with the finer particles potentially being washed out. This can lead to settlement, and further opening of joints or cracks; with increasing leakage flows. Left unchecked, the process may not be self-limiting, and very large leakage, leading to a breach mechanism by unravelling or instability of the downstream slope may result. At what stage this may occur is dependent on the zoning and the nature of the rockfill; e.g. gravel fill, and low strength, weathered rockfill, is likely to have lower permeability and be more likely to unravel or reach an unstable condition than other rockfill.

The one failure of a modern CFRD (Gouhou dam, Chen, 1993) occurred at apparently relatively small leakage, but the zoning of that dam was poor, with the fill being silty sandy gravel without high permeability drainage zones.

Hydro Tasmania, who owns and operates a number of CFRD, has developed a framework for quantifying the likelihood of failure of CFRD. In a 'workshop' held in October 2001 Richard Herweynan and Paul Southcott from Hydro Tasmania, and Robin Fell (Southcott *et al.*, 2003) concluded that the failure path for a CFRD could be likened to an internal erosion and piping failure for earth and rockfill dams, with the face slab and the joint system being equivalent to the earth core, Zones 2D and 2E the filters. Hence the failure path is as shown in Figure 15.50. The method is therefore strongly based on Foster (1999), Foster and Fell (1999b, 2000) and Fell *et al.* (2000).

Table 15.14 Data on leakage rates observed in early CFRD during first-filling and during operation.

			First Filling		*During Operation*				
Dam Name	*Height (m)*	*Water Depth at Full Supply Level (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Max Leakage Rate (l/sec)*	*Water Depth (m)*	*Long-Term Rate (l/sec)*	*Comments on Leakage Trends*	*Comments*
Cogswell	85.5	Unknown	3510	63.5	Unknown	Unknown	Unknown	Significant leakage on first filling due to face slab cracking as a result of large embankment settlements.	Construction involved dry dumped angular rockfill with later sluicing. Remedial works included placement of timber facing over face slab.
Courtright	96.5	94	Unknown	Unknown	1275	94	Unknown	Progressive increase in leakage rate measured over 9 yrs after EOC.	Construction involved dumping and high pressure sluicing of angular rockfill. Significant remedial works implemented to reduce leakage. Leakage related to face slab cracking and opening of perimetric joints.
Dix River	84	76	Unknown	Unknown	2700 (12 yrs after EOC)	Unknown	2000	Max leakage related to slab joints and perimetric joints.	Construction involved dumping of rockfill with sluicing of limited effect. Long-term rate measured 30 years after end of construction.
Lower Bear No. 1	75	71.5	110	71.5	85	71.5	25–40	Fluctuations in leakage rate closely related to reservoir level. Min rate 25 l/s at water depth less than 50 m.	Construction involved dumping and high pressure sluicing of angular, high strength rockfill. Long-term rate established approx two years after first filling.

Salt Springs	100	97.5	850	Unknown	425–565	97.5	Unknown	Fluctuations in leakage rate closely related to reservoir level.	Embankment construction involved dumped and poorly sluiced rockfill. Cracking occurred in the face slab before first filling.
Strawberry	43.5	Unknown	Unknown	Unknown	285	Unknown	170	Leakage slowly increased over 80 years of service.	Concrete facing replaced 12 yrs after EOC. Facing and joints have been repaired several times over the life of the dam.
Wishon	90	89	3120	89	Unknown	Unknown	850	Seasonal leakage behaviour is unknown.	Construction involved dumping and high pressure sluicing of rockfill. Maximum leakage occurred 2 months after first filling due to cracking in face slab.

Table 15.15 Factors to be considered, and methods for assisting in the estimation of the likelihood of initiation of a concentrated leak in CFRD (Southcott *et al.*, 2003).

Initiator	*Factors to consider*	*Methods for assisting in the estimation of likelihood of concentrated leak*
(A) Factors applicable to all load conditions		
Perimetric joint opens	• Crest and face slab deformation, first fill and continuing	Compared to population, and degree of variation across dam
	• Joint meter and face slab inclinometer displacement	Compared to design displacement and to population
	• Deep plinth, and local deep fill at plinth in over-break and steep plinth sections	Calculate joint displacement from depth, construction modulus, for zone 3A or modulus of zone 2F, allowing for water loading (depth)
	• Leakage flow rates vs reservoir level and time. If there is increasing leakage, or >50 l/sec? Maybe be a leak, >1000 l/sec certainly a leak	If no leakage measurements are available, use a range of values of likelihood to reflect the uncertainty
	• Water stop deterioration, accounting for water stop type, acidity of water, displacement	
Vertical joints open	• Crest settlement, horizontal displacement and face slab deformation first fill and continuing	Compared to population, and degree of variation across dam
	• Joint meter and face slab inclinometer displacements	Compared to design displacement and to population
	• Valley profile – e.g. steep valley sides leading to large differential settlements	Estimated total settlement and surface profile
	• Opening of crest wall joints (transverse)	If no leakage measurements use a range of value
	• Leakage flow rates vs reservoir level and time. If there is increasing leakage or >50 l/sec? May be a leak, >1000 l/sec certainly a leak	
Face slab cracking	• Crest and face slab deformation, first fill and continuing	Compared to population
	• Staged rockfill and/or staging of face slab construction effects, with potentially different modulus for lower and upper part	Construction data, calculation from rockfill depth, construction modulus, allowing for water loading (depth) and link to total deformation
	• Deep plinth, and local deep fill at plinth in over break and steep plinth sections	Construction data, from rockfill depth, construction modulus, allowing for water loading (depth) calculation and link to total deformation
	• Location of and nature of difficult construction joints	If no leakage measurements are available use a range of values of likelihood to reflect the un certainty
	• Leakage flow rates vs reservoir level and time. Increasing leakage or >50 l/sec? Maybe a leak, >1000 l/sec certainly a leak	
	• Crack mapping of face slab	
	• Sudden changes in slab thickness	
Crest wall joint opens	• Visual inspection/displacement on joint	
	• Design detail	
Piping under plinth	• Part or all of plinth foundation erodible, e.g. soils, completely weathered rock, clay infill joints	As detailed in Chapter 8.
	• Hydraulic gradient	
	• Grouting – curtain	
	• Grouting – consolidation	

(continued)

Table 15.15 Continued.

Initiator	*Factors to consider*	*Methods for assisting in the estimation of likelihood of concentrated leak*
(B) Additional factors applicable to flood loading		
All initiators	• AEP of flood levels	Hydrological analysis
	• Historic high flood level	Likelihoods much less for flood levels below historic high than above
	• Increased face slab deformations, joint openings, under flood loading	Calculate from dam geometry, properties, existing deformations and openings allowing for added water load
(C) Additional factors applicable to seismic loading		
All initiators	• Earthquake magnitude, PGA, vs AEP	Swaisgood (1998), Pells and Fell (2003) methods
	• Additional settlement induced by earthquake	Judgement/other analyses
	• Normal operating load case assessment of $P_{initiation}$	
	• Crest wall geometry + geometry re spillway wall	
	• Are there liquefiable (not necessary flow liquefaction) zones in the foundation. What is the AEP of liquefaction, and will freeboard be lost if liquefaction occurs	Conventional methods based on Standard Penetration Test or Cone Penetration Test, earthquake magnitude, AEP of PGA. Check post liquefaction factor of safety/deformations. See Chapter 12.

Table 15.16 Factors to be considered and methods for assisting in the estimation of the likelihood of continuation and progression of a concentrated leak in CFRD.

Initiator	*Factors to consider*	*Methods for assisting in the estimation of likelihood of concentrated leak*
(A) Continuation of concentrated leakage		
Perimetric joint opens	• As constructed particle size distributions and filtering characteristics of zones 2F, 2D, 2E, 3A (depending on zoning, allowing for segregation).	Assess filters against no-erosion, excessive erosion and continuing erosion criteria (Foster and Fell, 2001); Assess for internal instability (Wan and Fell, 2008). Chapter 9.
Vertical joint opens, face slab cracks, crest wall joint opens	• As constructed particle size distributions and filtering characteristics of zones 2D, 2E, 3A.	As above
Piping under plinth	• Presence of extent of filter(s) under rockfill downstream of plinth. If present, particle size of filter and erodible material in the foundation	As above
All joints, cracks	• Displacement/disruption of zone 2D by seismic loading	Judged from estimate of overall settlement using Pells and Fell (2003) or other methods. Chapter 12.

The likelihood of failure can be assessed qualitatively or quantitatively in this approach. In either case, the failure paths should be considered for first filling, under normal operating conditions, flood (up to dam crest level) and under seismic loads. The probability that the reservoir level will be exceeded in any year needs to be incorporated into any quantification of the normal operating, flood and seismic conditions.

For an overall risk assessment, the likelihood of failure by overtopping under flood; scour/erosion of the dam due to overtopping of spillway chute walls and spillway approach walls, and from waves caused by landslides into the reservoir should be considered.

This discussion concentrates only on those failure paths covered by Figure 15.50.

Clearly, what is important to any assessment of the likelihood of failure is the ability of the owner of the dam to respond to the leakage incident, by lowering the water in the reservoir to below the cracked zone, or by relieving the head driving the seepage and/or to 'seal' the crack or opening by, for example, placing sand on to the leak and have it wash into the crack, and be prevented from eroding further by filter action on Zone 2E, or Zone 2F.

15.8.2 Assessment of likelihood of initiation of a concentrated leak

Table 15.15 summarizes the factors to be considered for assessing the likelihood of initiation of leakage under normal operating and flood (to dam crest level) conditions. Similar factors would apply to first filling, but in that case the overall historic performance could be used to assist in quantifying the likelihood.

Also detailed are suggestions on the likelihood of initiation of a concentrated leak. The term 'compared to population' suggests the measurable value, e.g. slab displacement, can be compared to the data in Section 15.6, and for example, if the measured value is large compared to the population, a higher likelihood might be assigned. It will be apparent that the estimate will be highly subjective and difficult to quantify. Some points should be kept in mind:

- It is very unlikely a concentrated leak will form under normal operating conditions (water levels within annual levels), unless for example a perimetric joint is right at the limit of its displacement or there is a deteriorating condition of the joint filler or water stop measures.
- It is more likely that a leak would form under reservoir water levels higher than the historic high (because the loads will be greater than the dam has experienced before/or under large seismic loads (because of the added settlements induced by the earthquake, and/or damage to the face slab).

Southcott *et al.* (2003) give the conditional probabilities used by Hydro Tasmania in their risk assessment.

15.8.3 Assessment of the likelihood of continuation of a concentrated leak

Table 15.16 summarizes the factors to be considered, and methods for assisting in estimation of the likelihood of continuation of a concentrated leak. The factors affecting

Table 15.17 Factors to be considered and methods for assisting in the estimation of the likelihood of progression of piping in CFRD (Southcott *et al.*, 2003).

Initiator	*Factors to consider*	*Methods*
All initiators	• Zoning and particle size distribution of each zone, and potential for internal instability in zones 2D, 2E, 3A, 3B leading to washing out of fines	Assess for internal instability using Wan and Fell (2008). Section 8.5.
	• Flow rate and location of the initial concentrated leak	Judged from the initiating mechanism, precedent in other CFRD.
	• Potential for downstream face protection to intercept eroding material, leading to slowing down of internal erosion	Assess as a filter against the eroding fines for no-erosion, excessive erosion and continuing erosion criteria (Foster and Fell, 2001); Chapter 8.
	• Presence of zone 1A and 1B upstream of the plinth, and whether it will act to seal on zones 2F, 2D	As above.
	• Ability to intervene to lower the reservoir level, or seal the opening in the face slab or joints	Account for location and capacity of outlet works, reservoir volume, river inflows; accessibility of site, availability of barges, personnel, whether the joint or opening is covered by debris, zone 1A etc.
	• Potential for erosion, and formation of a pipe in alluvium left in the dam foundation	Judged, accounting for the initiating mechanism, downstream toe details, whether the alluvium "day lights" downstream of the toe, whether flow will be in alluvium or in dam fill.

continuation are relatively readily quantifiable using the methods outlined in Foster and Fell (2001) and Foster *et al.* (2002).

In this assessment, the particle size distribution of Zones 2D versus 2E, 2E versus 3A, and 2A versus 3A have to be assessed against the no erosion, excessive erosion and continuing erosion criteria in Foster and Fell (2001) and Foster *et al.* (2002). However, since in many existing dams the Zones 2D and 2E will be internally unstable, assessed using the criteria of Wan and Fell (2008), it is necessary to consider first whether the zone is internally unstable, and if so, what size particles will be washed from the zone through the adjacent zone before the adjacent zone will eventually act as a filter.

In many older dams, this may show that quite extensive internal erosion may occur before the filter action works, and the leakage rate may be quite high.

15.8.4 Assessment of the likelihood of progression to form a pipe

Table 15.17 lists the factors to be considered. In the case of CFRD it is unlikely a hole or pipe will form as would happen in an earthfill core. Rather, what is likely to happen is a continuing erosion of the finer fraction from the zones in the dam, including the rockfill. The erosion of the finer material will cause settlement, a likely greater opening of the joint or crack which initiated leakage, and so on unless there is intervention.

Table 15.18 Factors to be considered and methods for assisting in the estimation of the likelihood of formation of a breach in CFRD.

Breach and Mode	*Factors to consider*	*Methods for assisting in the estimation of likelihood of formation of a breach*
Slope instability and unravelling	• Estimated likely flow rate of the leak	Initiator mechanism, continuation and progression factors, ability to intervene to seal leak; historic precedents for modern and older CFRD (section 18.6).
	• Ability to intervene, draw down the reservoir	Account for location and capacity of outlet works, reservoir volume, river inflows; accessibility of site, availability of barges, personnel, whether the joint or opening is covered by debris, zone 1A etc.
	• Presence of alluvium in the foundations	Consider in relation to the breach mechanism.
	• Reservoir level at the time of the concentrated leak, freeboard above that level, and the ability	Assess from hydrology.
Unravelling	• Discharge capacity of rockfill zones, compared to estimated leakage flow rates, giving gradients through the dam	Estimate discharge capacity from zoning, particle size distribution, presence of steel meshing on downstream face and methods for assessing unravelling of rockfill using the methods described in Section 8.11. Estimate leakage flow rates through the concrete face by considering potential crack or joint openings and estimate flows through the crack using ICOLD (2010) or Kanitkar *et al.* (2011). Estimate the flow limiting effects of the Zone 2D using flow nets as described in Sherard (1985b).
Slope instability	• Discharge capacity of rockfill zones compared to estimated leakage flow rates, giving pore pressure rise in the dam	As above. Calculate factors of safety, property allowing for non-linear strengths of the rockfill materials, and high friction angles at low confining stress.
	• Effect of downstream slope protection/steel meshing, which may act as a filter to eroding material, causing pore pressures to rise	Assess as a filter against the eroding fines for no-erosion, excessive erosion and continuing erosion criteria (Foster and Fell, 2001) Chapter 8.

15.8.5 Assessment of the likelihood of a breach forming

Table 15.18 lists the factors to be considered and the methods which can be used to calculate whether for example, unravelling will occur. The ability to intervene to stop the process by drawing down the reservoir, or reducing the leakage by clogging it with silty-sandy-gravel (for example) can be assessed depending on the particular circumstances of the dam. The erosion process is not likely to be so rapid as for earth and earth and rockfill dams, but may still progress in days or weeks, so detection and intervention need to be done soon after the significant leak develops.

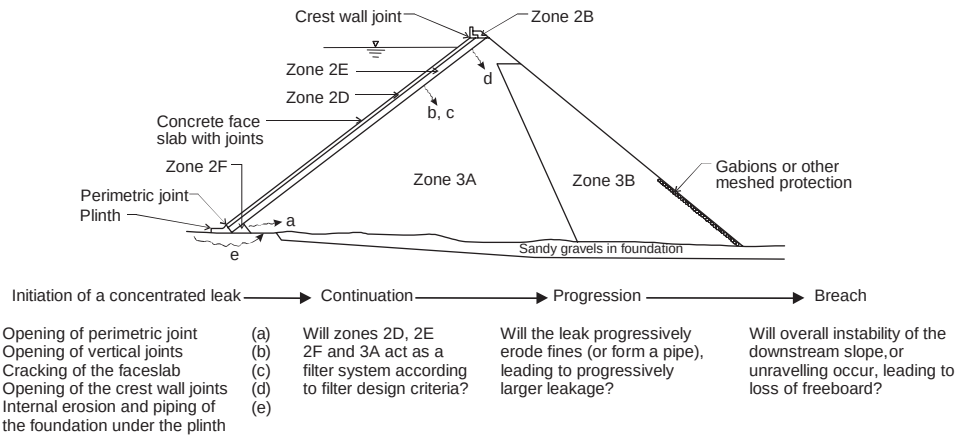

Crest wall joint
Zone 2B
Zone 2E
Zone 2D
d
b, c
Concrete face slab with joints
Zone 2F
Zone 3A
Zone 3B
Gabions or other meshed protection
Perimetric joint
Plinth
a
e
Sandy gravels in foundation
Initiation of a concentrated leak
Continuation
Progression
Breach
Opening of perimetric joint (a)
Opening of vertical joints (b)
Cracking of the faceslab (c)
Opening of the crest wall joints (d)
Internal erosion and piping of the foundation under the plinth (e)
Will zones 2D, 2E 2F and 3A act as a filter system according to filter design criteria?
Will the leak progressively erode fines (or form a pipe), leading to progressively larger leakage?
Will overall instability of the downstream slope,or unravelling occur, leading to loss of freeboard?

Figure 15.50 Failure paths for concrete face rockfill dam.

15.8.6 Concluding remarks

It will be apparent that what is described above is only a framework, and quantification will be difficult and subject to considerable uncertainty. It may be sufficient in many cases only to use qualitative terms, as was done by Bell *et al.* (2002) for the assessment of the likelihood of piping in Eucumbene earth and rockfill dam, the process of formally considering all failure initiators and failure paths is a valuable one, even if the estimates of likelihood are approximate.

These should only be considered very approximate as they are based on engineering judgement and a "mapping scheme" linking probability in qualitative and quantitative terms, with little data to aid in quantification.

15.9 FURTHER READING

This chapter has set out to summarize the history of development, features, and performance of CFRD. Those seeking more detail should refer to the following volumes:

J. Barry Cooke Volume, Concrete Face Rockfill dams. Chinese Committee on Large Dams, Beijing, September 2000.

Proceedings, International Symposium on Concrete Face Rockfill Dams, September 2000, International Commission on Large Dams, Paris.

Proceedings, Second Symposium on Concrete Face Rockfill Dams, Comite Brasileiro de Barragens, Rio de Janeiro, October 1999.

Concrete Face Rockfill Dams, P.T. Cruz, B. Materon and M. Freitas. CRC Press, 2009.

ICOLD Bulletin 141, Concrete Face Rockfill Dams, Concepts for Design and Construction. ICOLD, Paris.

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Chapter 16

Concrete gravity dams and their foundations

16.1 OUTLINE OF THIS CHAPTER

This chapter presents the basics of the analysis of the stability of concrete gravity dams on rock foundations using two dimensional rigid body analysis methods. It is based largely on four texts.

Canadian Dam Safety Association, *"Dam Safety Guidelines"* (CDSA, 1999).
BC Hydro, *"Guidelines for the Assessment of Rock Foundations of Existing Concrete Gravity Dams"* (BC Hydro, 1995).
Federal Energy Regulatory Commission, *"Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 111, Gravity Dams"* (FERC, 2002).
Australian National Committee on Large Dams, *"Guidelines on Design Criteria for Concrete Gravity Dams"* (ANCOLD, 2013).

This Chapter deals mainly with the normal operating and flood load cases, with a brief outline of analysis of the earthquake loading case.

As detailed in Douglas *et al.* (1998, 1999), and recognized in CDSA (1999), BC Hydro (1995) and ANCOLD (2013), the most common mode of failure of gravity dams is for sliding or overturning in the foundation, so there is a discussion here on how to estimate the shear and tensile strength of the rock in the dam foundation. There is also a discussion on the estimation of uplift pressures and the shear and tensile strength of the concrete in the dam.

16.2 ANALYSIS OF THE STABILITY FOR NORMAL OPERATING AND FLOOD LOADS

16.2.1 Design loads

The following design loads are usually considered in the analysis of stability. Most are shown in Figure 16.1.

Dead loads (D), comprising the weight of the concrete plus the weight of spillway bridges, gates, piers and attached intake structures.

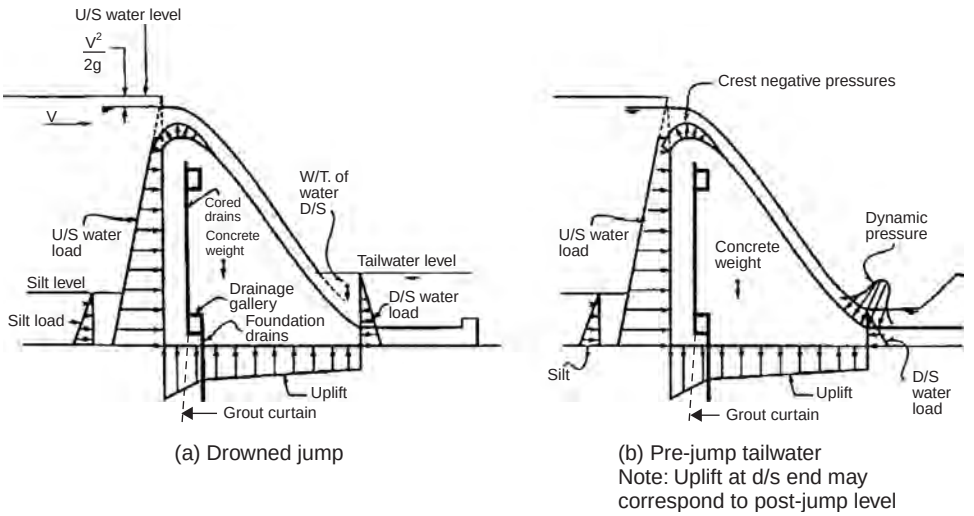


Figure 16.1 Design loads on gravity spillway (adapted from ANCOLD, 1991).

Water loads (H, H_F), consisting of:

(a) Horizontal load on the upstream face.
(b) Horizontal load from the tailwater.
(c) Vertical loads on the upstream face, if it is sloping, or behind crest gates.
(d) Pressures from the water flowing over a spillway section. These will also have a vertical component.

H is taken here as the loads at maximum normal reservoir level, combined with the most critical tailwater level, and H_F is the load at maximum flood reservoir level based on the inflow design flood (IDF) with corresponding water levels.

Internal water pressure and uplift (U), which act within the rock foundation, at the foundation-dam interface and within the body of the dam.
Static and dynamic thrust (I), created by an ice sheet for reservoirs subject to freezing.
Silt and earth load (S), vertical and horizontal loadings due to soil or rock backfill and from silt deposited against the structure.

Maximum design earthquake load (Q).
Estimation of these loads is discussed below.

16.2.2 Load combinations

The following load combinations should be considered (adapted from CDSA, 1999).

Usual Loading. Permanent and operating loads should be considered for both summer and winter conditions including self-weight, ice, silt, earth pressure and the normal

maximum operating water level with appropriate uplift pressures and tailwater level ($D + H + I + S + U$).

Unusual Loading (blocked drains). Loads should be as for the usual loading case, except that the drains should be assume inoperative ($D + H + I + S + U_{BD}$).

Unusual Loading (post earthquake). If earthquake induced cracking is identified at the rock-concrete interface or at any potential kinematically viable failure surface in the dam or foundation, this may result in altered uplift pressures and/or reduced strength of the concrete or foundation. A stability analysis should be carried out to see whether the structure in its post-earthquake condition is still capable of resisting the usual loading ($D + H + S + U_{PQ}$). Concurrent ice loading should be considered in areas where appropriate.

Unusual loading inoperative drains post earthquake It may be necessary to consider an inoperative drain case post earthquake if the deformations resulting from the earthquake are sufficient to displace the dam on its foundations and make the drains inoperative. In some cases, e.g. where large displacements are expected at the dam-foundation contact, this may be a highly likely condition given the earthquake occurs.

Flood Loading. Permanent and operating loads of the Usual Loading, except for ice loading, should be considered in conjunction with reservoir and tailwater levels and uplift resulting from the passage of the inflow Design Flood (IDF) ($D + H_F + S + U_F$).

The effect of ice loads is not usually considered simultaneously with design flood conditions but that should be assessed for the dam being analysed taking account of climatic conditions.

Earthquake Loading. If pseudo static analyses are being carried out, permanent and operating loads from the Usual Loading case should be considered in conjunction with the seismic loads of the Maximum Design Earthquake (MDE) or Safety Evaluation Earthquake (SEE); ($D + H + S + Q + U_Q$). The effects of ice should be given special consideration, recognizing the high uncertainty associated with ice loading on earthquake loading and its effects on the structure.

In the above, subscript '$_{BD}$' refers to the blocked drains case. Subscript '$_{PQ}$' to the post-earthquake case and subscript '$_F$' to the flood case.

Temperature induced loads would normally be ignored in this level of analysis, although they may be significant if analysing stresses in a finite element analysis.

16.2.3 Kinematically feasible failure models

It is absolutely essential that there be a proper assessment of the geological structure in the dam foundation and, based on this the kinematically feasible failure modes should be identified.

Examples of typical, and not so common failure modes are given in Figures 16.2 and 16.3 respectively.

It should be noted that there may be different failure modes in different parts of the dam foundation.

A major failing of dam engineers has been considering only the mode of failure involving shear along the concrete-rock contact. In most dams there may be more

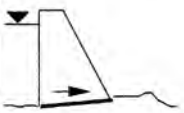
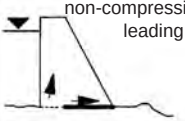
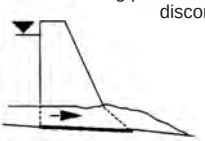
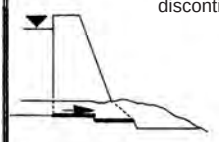
Mode	Condition	Comments
1. Shear along concrete rock contact 	- Shear along weak discontinuity at or near concrete-rock contact.	- Possible if weak foundation or poor excavation and/or foundation treatment. - Infrequent mode. Check for condition, particularly in old dams.
2. Development of non-compression zone leading to shear 	- Development of non-compression zone (a computed crack) leading to shear along the concrete-rock contact.	- Can occur even in good foundations.
3. Shear along planar discontinuity 	- Shear along planar or near planar discontinuity in rock foundation.	- Frequent mode.
4. Shear along stepped discontinuities 	- Shear along stepped discontinuities or along surfaces stepping down between parallel discontinuities in rock foundations.	- If step(s) not large then failure surface may be approximated by an inclined plane. - Large steps may require higher level of analysis.
5. Shear through jointed rock mass 	- Shear through highly jointed rock mass (both natural or blast induced) or through weak rock mass.	- Infrequent mode - May be used in moderately jointed rock masses (with an adjustment to assigned Hoek-Brown rock mass strength) - Failure surface usually approximated as being planar.

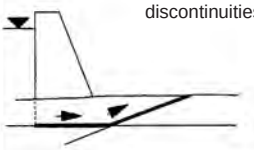
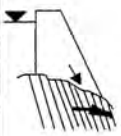
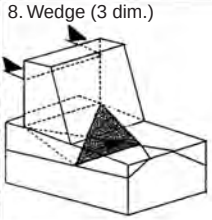
Figure 16.2 Typical kinematically feasible failure modes (BC Hydro, 1995).

critical conditions involving shear along discontinuities in the rock foundations if these have a lower strength than the concrete-foundation interface.

The analysis should also consider failure within the concrete in the dam, usually controlled by the most likely distinctive weaknesses within the concrete, the construction lift joints. In some dams there will be identifiable weak planes due to deterioration on lift joints and cracking due to thermal expansion and contraction on major discontinuities in the geometry of the dam.

16.2.4 Analysis of stability

For most structures it is sufficient to analyse the stability using rigid body force equilibrium analysis. The analysis should be carried out for failure within the dam, at the

Mode	Condition	Comments
6. Shear along combined discontinuities 	- Shear along a combination of two or more discontinuities.	- Occasional; usually assume planar mode (3) during initial assessment. - Limit equilibrium analysis can be used for more detailed analysis.
7. Toppling 	- Loads at toe of dam causing toppling of bedded rock formation.	- Infrequent but should be evaluated in bedded or highly jointed (parallel) rocks - If dam heel lifts block may continue movement by shearing through toe (mode 2.).
8. Wedge (3 dim.) 	- Wedge formed beneath block by combinations of major faults, shears and/or joint sets.	- Infrequent but should be evaluated where major features such as faults/shears cross foundation and combine with jointing. - Rigid body, 3-dimensional analysis.

Note: Failure modes such as these require a detailed analysis in a higher level assessment than covered by these guidelines.

Figure 16.3 Other kinematically feasible failure modes (BC Hydro, 1995).

dam-foundation contact and for critical kinematically feasible failure surfaces in the foundation. For failure in the foundation it will be necessary to carefully consider the uplift pressure and the resistance the foundation can provide to sliding.

In the absence of more detailed modeling, it is suggested that the assumptions recommended by BC Hydro (1995) be adopted. These are shown in Figure 16.4 and are:

(a) Water pressure due to the reservoir exists on a surface extending downward from the heel of the dam to the potential failure surface. This pressurized surface shall be assumed to be vertical unless adverse orientation of foundation joints or faults results in a more onerous assumption.

(b) No resistance to loading from the foundation exists beyond a line oriented downward at 45° from the dam's toe.

(c) Uplift pressures are either estimated as outlined in Section 16.6, or consistent with observed piezometric conditions, provided instrumentation is adequate and drainage is well maintained. Bounding pressures are:

  (i) Reservoir pressure at the intersection of the potential failure surface and the surface extending downward from the heel of the dam.

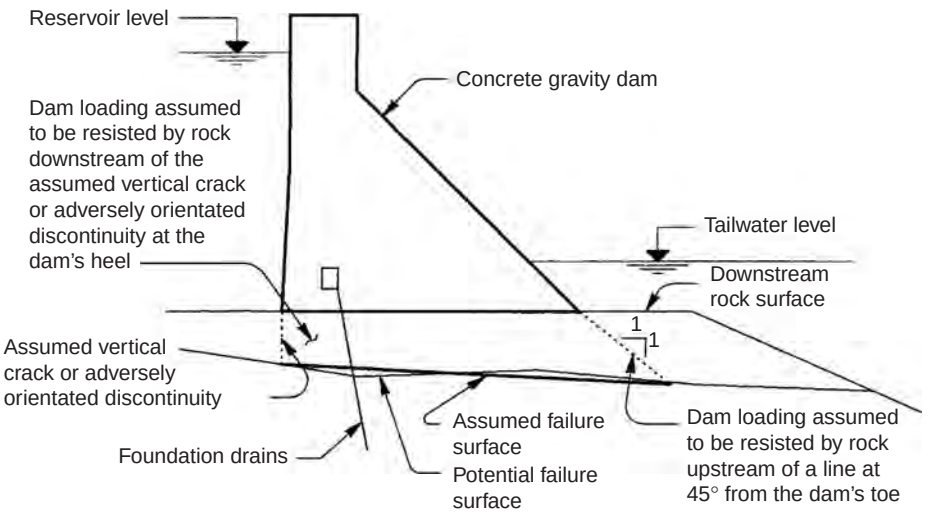

Reservoir level
Concrete gravity dam
Dam loading assumed to be resisted by rock downstream of the assumed vertical crack or adversely orientated discontinuity at the dam's heel
Tailwater level
Downstream rock surface
1
1
Assumed vertical crack or adversely orientated discontinuity
Assumed failure surface
Potential failure surface
Dam loading assumed to be resisted by rock upstream of a line at 45° from the dam's toe
Foundation drains

Figure 16.4 Assumptions for analysis of potential failure surfaces in the foundation (BC Hydro, 1995).

(ii) Pressure due to tailwater or due to a piezometric surface at the elevation of the ground surface downstream of the dam at the intersection of the potential failure surface and the 45° line from the dam's toe. The potential for artesian pressures in the vicinity of the dam's toe due to causes such as limited drainage, anisotropic permeability or lateral groundwater flows, should be evaluated.

(d) There is no tensile strength on the potential failure surface or the surface extending downward from the heel of the dam.

Rigid body analysis of static stability on a plane in the foundation consists of:

(a) Vector summation of all forces including uplift acting on the foundation and dam above the assumed failure surface into force components normal and parallel to the failure surface.
(b) Determination of the resultant force location on the assumed failure surface from the summed moments due to all forces acting on the foundation and dam above the surface and the normal component of the resultant force.
(c) Calculation of stresses normal to the assumed failure surface from the normal component of the resultant force and its position on the surface, assuming a linear variation of stress.
(d) Adjustment of the assumed uplift distribution to reservoir pressure where tensile normal stresses greater than the available tensile strength are found and iterative recalculation of the resultant force and the normal stresses until the assumed and calculated amount of the surface in compression correspond. For failure in the foundation, the rock will have zero tensile strength.

(e) Calculation of the stability indices using the resultant force components. These are the sliding factor (calculated using peak and residual strengths in the concrete or foundation), the position of the resultant force and the compressive strength factor in the concrete and foundation. The Sliding Factor (SF) is defined as:

$$\text{SF} = \frac{\text{Available Shear Resistance}}{\text{Net Driving Force}} \tag{16.1}$$

The net driving force represents the collection of tangential components of all forces acting above the sliding surface (in the direction considered).

Two states of available shear resistance of concrete and the foundation should be considered: peak and residual.

The peak shear resistance should be based on the following components:

(a) Normal Stress (Sn) acting on the potential sliding surface
(b) Peak angle of effective internal friction (ϕ')
(c) Area of compression (Ac)
(d) Threshold shear strength (τ_o) at zero normal stress (sometimes called the 'cohesion')

and calculated from:

$$\text{Available Peak Shear Resistance} = \Sigma \text{Ac}\{(\text{Sn}) \tan(\phi') + \tau_o\} \tag{16.2}$$

The residual or post-peak resistance should be based on the following components:

(a) Normal Stress (Sn)
(b) Residual angle of effective sliding friction (ϕ'_R)
(c) Area of compression (Ac)
(d) Nominal residual threshold shear strength value (τ_{nr}) up to 100 kPa if supported by tests. Without tests, τ_{nr} should be considered to be zero and the available residual shear strength and calculated from:

$$\text{Available Residual Shear Resistance} = \Sigma \text{Ac}\{(\text{Sn}) \tan(\phi'_R) + \tau_{nr}\} \tag{16.3}$$

The compressive strength factor is given by:

$$\text{compressive strength factor} = \frac{\text{unconfined compressive strength}}{\text{maximum compressive stress normal to the assumed failure surface}} \tag{16.4}$$

The compressive strength factor is considered primarily for massive but low strength rock and weak weathered concrete. In applying the formula to jointed rock foundations, the unconfined compressive strength of the jointed rock should be assessed based on the ultimate bearing capacity as described in Section 16.3.5.

Table 16.1 Suggested acceptance criteria for position of resultant force and compressive strength factor for gravity dams (adapted from CDSA, 1999 and BC Hydro, 1995).

	Load Case			
	Usual	*Unusual post-earthquake and blocked drains*	*Earthquake (MDE)*	*Flood (IDF)*
Position of Resultant Force	Inside Middle 1/3rd		Outside middle 1/3rd if other acceptance criteria satisfied	
Compressive Strength factor				
(a) concrete	3	1.5	1.1 preferably 1.3	2.0
(b) foundation(1)	4	2.0	1.3	2.7

Note: (1) To be considered primarily for massive but low strength rock and weak weathered concrete. For jointed rock foundations apply these factors to the ultimate bearing capacity assessed as described in Section 16.3.5.

The assessment of the stability should also consider observed behaviour and monitoring data such as ambient, water and concrete temperatures, plumb line deflections, joint meter readings, surface monument displacements, foundation piezometric pressures, foundation and structure drain flows, extensometer readings and strong motion accelerograph records.

16.2.5 Acceptance criteria

The suggested acceptance criteria given in Tables 16.1 and 16.2 are based on those from CDA (1999) and BC Hydro (1995) and discussions arising from the development of ANCOLD (2013) guidelines. They are considered a reasonable and logical approach.

The following are qualifications to Table 16.2.

(1) Advice from an experienced engineering geologist and rock mechanics person(s) should be sought to assess kinematically feasible failure surfaces and the foundation strength. The factors of safety are predicated on the adoption of reasonably conservative strengths. For the residual strength condition the adopted strength should be at or near the lower bound of good quality test data and assume no rock bridges are effective.

(2) "Not well defined" means that for an existing or proposed new dam there is limited exposure of the rock in the foundation at the dam site; there are limited data on foundation geology from site investigations or from construction records for an existing dam of foundation geology including information on bedding and defects in the rock and no or few strength tests. *Where there is no data, no reasonable assessment can be made and data must be obtained if the assessment is to be meaningful.*

(3) "Well defined" means that for an existing or proposed new dam there is good exposure of the rock in the foundation at the dam site; there are ample data from

Table 16.2 Suggested acceptance criteria for sliding factors for gravity dams (adapted from CDSA, 1999 and BC Hydro, 1995). Read in conjunction with notes in the text below.

	Load Case			
Type of analysis	*Usual*	*Unusual (Post-earthquake, or drains blocked)*	*Earthquake (MDE)*[b]	*Flood (IDF)*
Peak Sliding Factor in Concrete (PSF) – Not well defined	3.0	2.0	1.3	2.0
Peak Sliding Factor in Concrete (PSF) – Well defined	2.0	1.5	1.1	1.5
Residual Sliding Factor in Concrete (RSF)[c] – Well defined	1.5	1.1″ preferably 1.3	1.0	1.3
Peak Sliding Factor in Foundation – Well defined	1.5 to 2.0	1.2 to 1.4	1.1 to 1.3	1.3 to 1.5

Notes: [a] PSF is based on the peak shear strength of the concrete or rock foundation. RSF is based on the residual or post-peak strength. See Section 16.3 and 16.4 for details.
[b] The stated value under the MDE load case is based on pseudo-static analysis. Performance evaluation of the dam should also take into consideration the time dependent nature of earthquake excitations and the dynamic response of the dam.
[c] If PSF values do not meet those listed above, the dam stability may be considered acceptable provided the RSF values exceed the minima.
[d] See text for description of "not well defined" and "well defined".

site investigations or from construction records for an existing dam of foundation geology including information on bedding and defects in the rock and sufficient strength tests to define strengths with confidence.

(4) Where the following conditions for "well defined" situations are met, sliding factors of safety at the low end of the range may apply.
   (i) There is no reliance on rock bridges and failure surfaces follow well defined bedding surfaces, shears or continuous joints.
   (ii) The effects of large and small scale asperities on the failure surface have been allowed for in a conservative manner based on good quality information.
   (iii) The shear strength assumed for design has no "cohesion" component although the strength envelope may be curved and so an apparent cohesion may apply to model the strength envelope.
   (iv) Foundation pore pressures for an existing dam are measured or will be confirmed by piezometers in the foundation for a new dam.

The actual factor of safety should be determined taking account of the quality of the information about the foundation, the degree of confidence in the strength and pore pressure estimates.

(5) Large-scale roughness or waviness on failure surfaces in the foundation (e.g. undulations in bedding surfaces) should not generally be modelled as rock

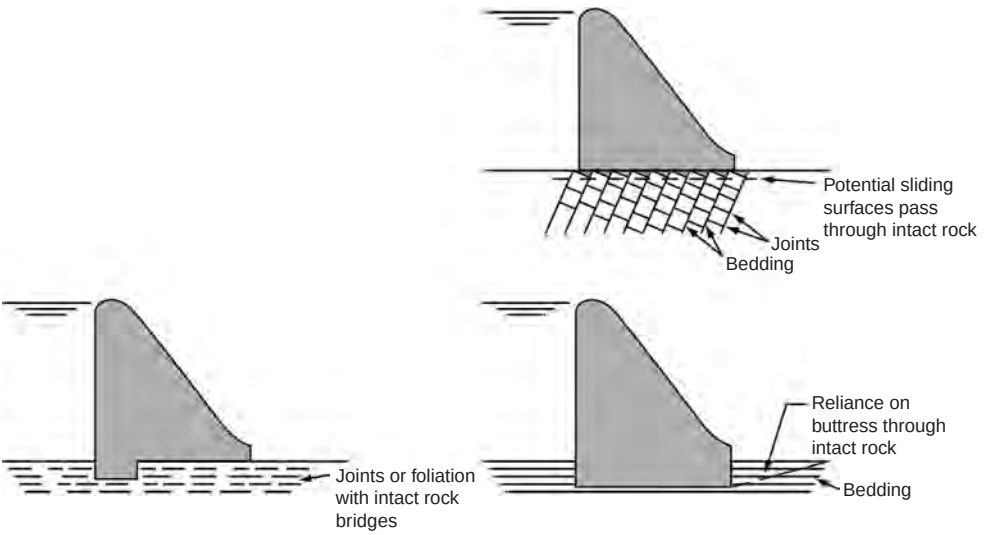


Figure 16.5 Some conditions where rock bridges form part of the potential failure surface.

bridges. To fail along such an undulating surface the failure block rides up toward the crest of the wavy surface rather than breaking through intact rock unless the stresses are so high the rock substance strength controls. It is usually treated by the $\phi'_b + i'$ approach as described in Barton and Bandis (1991). Rock bridges, and $\phi'_b + i'$ conditions may occur along a single feasible surface of rupture. In these cases strength is a composite of rock bridges and defects, allowing for strain compatibility and the higher factors of safety apply. Refer to Figure 16.5.

It should be noted that there are few dam foundations where Hoek-Brown criteria strengths will apply because at the scale of a dam foundation, the bedding, foliations and other defects will more commonly control kinematically feasible failure surfaces. If Hoek-Brown criteria strengths do apply they infer rock bridges are effective and the higher factors of safety apply.

(6) There will be foundations where tectonically and/or valley stress relief induced weak surfaces are present (e.g. bedding surface shears), and the actual strength will be the residual. In this situation, the 'peak strength' criterion should be met using this actual strength.

(7) In the post-earthquake load case, the sliding factors may have fallen below the 'Usual' FoS acceptance criteria but should still be not less than 1.2. In high consequence dams, this post-earthquake minimum factor of safety should be as close to 1.3 as is practicable. If for an existing dam the assessed sliding factor falls below 1.1 following an earthquake, there is a high likelihood of failure. In such cases the dam should be remediated to meet the sliding factor requirements for the 'Usual' load case.

(8) These criteria are intended to be applied to all dams regardless of the consequence of failure category.

Table 16.3 Recommended Design floods according to ANCOLD (2000b).

Load Cases	*Usual*	*Unusual*	*Extreme*
Extreme and High Consequence Category Dams	1:50 AEP flood but FSL as a minimum case	1:2,000 AEP flood	Extreme: PMF High A: PMP Design Flood (PMFDF) High B: 10^{-4} AEP to PMPDF or 10^{-6} AEP High C: 10^{-4} AEP to PMPDF or 10^{-5} AEP
All other Consequence Category Dams	As above	1:500 AEP flood	1:1,000 AEP to 1:10,000 AEP

It should noted that adequate sliding resistance is normally indicated by sliding factors which equal or exceed the minimum values listed in Table 16.2. For dams in relatively narrow valleys (width/height ratio less than about 3.0), beneficial three-dimensional effects could be present. If beneficial three-dimensional effects are demonstrable, the stated sliding factors may understate the stability of the dam. It is not common to rely on such 3D effects.

The minimum acceptable sliding factors, compressive strength factors, and resultant force locations in Tables 16.1 and 16.2 for the post-earthquake conditions are not intended for long term application. Thus, provisions should be made to inspect the dam promptly after a significant earthquake, to monitor its behaviour and to make any necessary repairs within a reasonable period of time or to clean blocked drains. The reservoir could be operated temporarily, if required, at a reduced level until repairs are made and/or safety of the dam is confirmed by analysis.

Table 16.3 gives suggested flood loading requirements to be used in the analyses. Table 12.2 gives the suggested earthquake loadings. Readers will need to comply with guidelines and regulatory requirements in their country and/or state or province as the case may be. The description of the consequence categories is given in Tables 10.3 and 10.4.

16.3 STRENGTH AND COMPRESSIBILITY OF ROCK FOUNDATIONS

16.3.1 Some general principles

The strength and compressibility of rock masses in the foundations of concrete dams can be assessed systematically by well-established engineering geological and rock mechanics principles.

Too often in the past, in the authors' experience, strength and compressibility were taken from guidelines. These, such as those provided in ANCOLD (1991), are often not conservative and potentially quite misleading. They may include large effective cohesion components to the strength which are simply not available in many situations and may, quite wrongly, suggest that the rock may reliably have some tensile strength.

It is therefore quite essential, even for relatively small structures, such as spillway crests, or gated spillway structures to engage the services of a geotechnical

professional(s) who has (have) knowledge of engineering geology and rock mechanics to assess the strength and, for larger projects, the compressibility of the rock foundations.

This assessment should include:

(a) Mapping of rock exposure, inspection of photographs taken during construction (for existing dams) and logging of drill core to determine the stratigraphy, bedding orientation, joint orientation, continuity and nature of the joint surfaces and to locate and delineate shears, faults, bedding surface shears and other features important to the strength.
(b) Carry out laboratory testing on representative samples of the rock to determine the unconfined compressive strength and basic friction angle (ϕ'_b) of the bedding surfaces, joints and other defects which may control stability.

    In preliminary assessments, or for some smaller dams, the basic friction angle may be assessed from published information, in which case conservative values should be adopted.
(c) Develop a geotechnical model of the foundation (i.e. plan, factual and interpretive sections) to define any possible kinematically feasible potential failure surfaces within the rock and/or on the foundation-rock contact. Use rock mechanics principles to determine the strength and compressibility of the foundation.

There will be variations in these conditions across the dam and it may be necessary to analyse the stability of several cross sections.

16.3.2 Assessment of rock shear strength

16.3.2.1 *General requirements*

The shear strength of the rock depends on the nature of the potential failure surface and the rock it passes through. The following approach to assessing the strength is based on BC Hydro (1995) and consultation with colleagues with a knowledge of rock mechanics.

BC Hydro (1995) who developed their guidelines with the assistance of rock mechanics and engineering geology experts Dr.'s E. Hoek and D. Deere, recommends the methods given in Table 16.4. The descriptive terms for discontinuities (defects) used by BC Hydro have been modified here to conform to the definitions in Chapter 2, Section 2.3 and Figure 2.1.

16.3.2.2 *Shear strength of clean discontinuities*

The shear strength of clean, rough (or smooth) discontinuities should be assessed using the approach of Patton (1966), who recommended use of the form of Equations 16.5 or 16.6.

$$\tau = \sigma'_n \tan(\phi'_b + i) \quad \text{if } \sigma'_n < \sigma'_{ns} \tag{16.5}$$

or

$$\tau = c'_a + \sigma'_n \tan \phi'_b \quad \text{if } \sigma'_n > \sigma'_{ns} \tag{16.6}$$

Table 16.4 Rock discontinuity shear strength parameters (adapted from BC Hydro, 1995).

Discontinuity Type	*Typical Feature*	*Strength Parameters (Note 1)*
Clean discontinuity (No previous displacement)	Clean joint, bedding surface	$\phi' = f(\phi'_b, i); c' = 0$
Thick infilled or extremely weathered seam (No previous displacement)	Infilled seam, joint, or extremely weathered bed	$\phi' = f(\phi')$ of infill or extremely weathered material (2); $c' = 0$
Discontinuity with previous Displacement	Sheared zone or seam, bedding surface shear, crushed seam	$\phi' = f(\phi'_r)$ of shear surface and/or sheared or crushed material, and f(i) of wall rock; $c' = 0$
Multiple discontinuity	Highly jointed rock mass	m_b, s, a, σ_{ci}

Notes: (1) Strength parameters as defined in Section 16.2:
f function of
c' effective cohesion (at zero normal stress)
ϕ'_b effective basic friction angle (for wet surfaces)
ϕ'_r effective residual friction angle
i average roughness angle
m_b, s, a Hoek-Brown criterion (1995) parameters
σ_{ci} uniaxial compressive strength of intact rock
(2) Test to be carried out on remoulded samples; ϕ' to be based on peak strength under drained conditions; c' to be neglected.

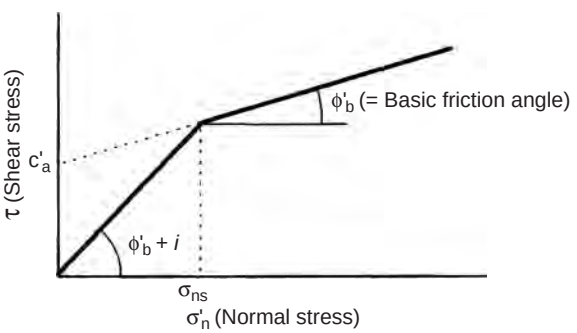


Figure 16.6 Shear strength of rough clean discontinuities in rock.

where $\sigma'_n = \sigma_n - u$ = effective normal stress; σ'_{ns} = effective normal stress at which the strength of asperities is exceeded; u = internal water pressure (pore pressure) within the discontinuity; ϕ'_b = basic friction angle (for wet surfaces) of the discontinuity wall rock; σ'_n = effective normal stress across discontinuity; i = average roughness angle; c'_a = effective apparent cohesion.

Figure 16.6 shows the application of the equations.

The basic friction angle (ϕ'_b) is derived from shear tests on sawn and lightly ground surfaces of the discontinuity (Barton, 1982; Barton and Bandis, 1991) or alternatively by measuring both vertical and horizontal displacements of a discontinuity specimen during laboratory direct shear testing and calculating the roughness angle i, which is then subtracted from the measured friction angle to determine ϕ'_b.

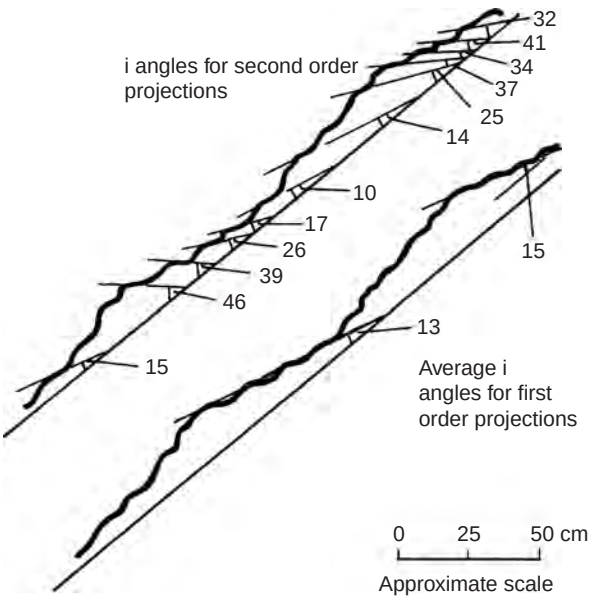


Figure 16.7 Measurement of roughness angles: (i) for first and second order roughness on a rock surface (adapted from Patton, 1996).

The roughness angle (i) is generally determined by field observations of surface roughness, at a scale appropriate to the dam foundation, in the direction of potential displacement (i.e. the first order roughness in Figure 16.7). BC Hydro (1995) recommend that for the purpose of dam engineering an upper bound of $i = 10°$ be applied. They also suggest that in dam foundations normal stresses tend to be low compared to the strength of the foundation rock so that for relatively hard (strong) rocks Equation 16.5 can be used. In soft (weak) rocks they suggest use of Equation 16.6 with $c'_a = 0$.

BC Hydro suggests that the bi-linear Patton (1966) model can be replaced with the curvilinear approach of Barton and Bandis (1991). The authors are satisfied that the Patton (1966) approach is adequate for most situations.

16.3.2.3 *Shear strength of infilled joints and seams showing evidence of previous displacement*

BC Hydro recommend that the shear strength of previously displaced features (e.g. sheared or crushed seams) is determined from:

$$\tau = \sigma'_n \tan(\phi'_r + i) \tag{16.7}$$

where ϕ'_r = effective residual friction angle of the infill, sheared or crushed material; i = average roughness angle which is reduced from the measured value to account for the lack of intimate contact of the walls of the feature as detailed below.

The residual frictional strength of the feature is generally determined by shear tests on undisturbed samples measured at sufficient displacement to ensure that the residual strength of the feature has been reached. For thick gouge (crushed rock) zones or, if undisturbed samples cannot be obtained, remoulded, normally consolidated samples of the material are tested for their residual strength. Gouge is often layered so care should be exercised to ensure that the weakest layer of material is selected for testing. Sufficient consolidation time must be allowed and rates of shearing must be sufficiently slow to allow pore pressure dissipation in accordance with accepted principles of soil mechanics. BC Hydro (1995) suggest the residual frictional component of shear strength for design be determined from the mean of the residual strength values if consistent results are obtained and a sufficient number of tests are done (about ten or more). However the authors' experience is that it is difficult to obtain true residual strengths and it is recommended more conservative values be adopted, e.g. the lower quartile or lower bound of test results.

To determine the roughness angle of previously sheared features, field measurements are obtained of the mean amplitude of asperities of the upper and lower surfaces of the shear, the mean inclination of asperity surfaces and the mean thickness of infilling. The value of roughness (i) will be less than the measured inclination of asperities because the walls of the feature are not in contact. Figure 16.8 indicates the reduced values of roughness for features with various ratios of seam thickness to asperity amplitude (from the USBR, based on Ladanyi and Archambault, 1977).

In many cases there may be a lower strength discontinuity (such as bedding surface shears or thin graphitic shears in sedimentary rocks) located along one of the contacts of the rock wall and seam material. BC Hydro (1995) recommend such features should be looked for and recorded during the geological data gathering process and tested separately from the bulk of the seam material, to allow the strength of the weakest element to be determined.

16.3.2.4 *Shear strength of thick infilled joints, seams or extremely weathered beds with no previous displacement*

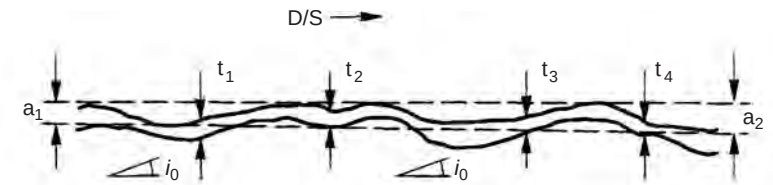
As indicated by BC Hydro (1995), such features would normally be excavated from the foundation of concrete dams. If they do exist, they suggest using:

$$\tau = \sigma'_n \tan(\phi') \tag{16.8}$$

where ϕ' = effective friction angle of the infilling or seam material, based on tests of remoulded samples. Any cohesion component c', indicated by testing, is usually very small and should be ignored.

16.3.2.5 *Shear strength of jointed rock masses with no persistent discontinuities*

Careful mapping of the foundation of most concrete dams will show that there are kinematically feasible failure mechanisms which are controlled by discontinuities (bedding, joints, shears, faults) which are persistent in the scale of the dam foundation. However for closely jointed rock with several sets of joints not oriented unfavourably in respect to potential failure surfaces; or for failure parallel to the base of the dam where the



Mean thickness = $(t_1 + t_2 + t_3 + t_4)/4$
Mean amplitude = $(a_1 + a_2)/2$ (in this case upper surface used)
Assuming no cohesion, increase friction angle
of filling by effective roughness angle

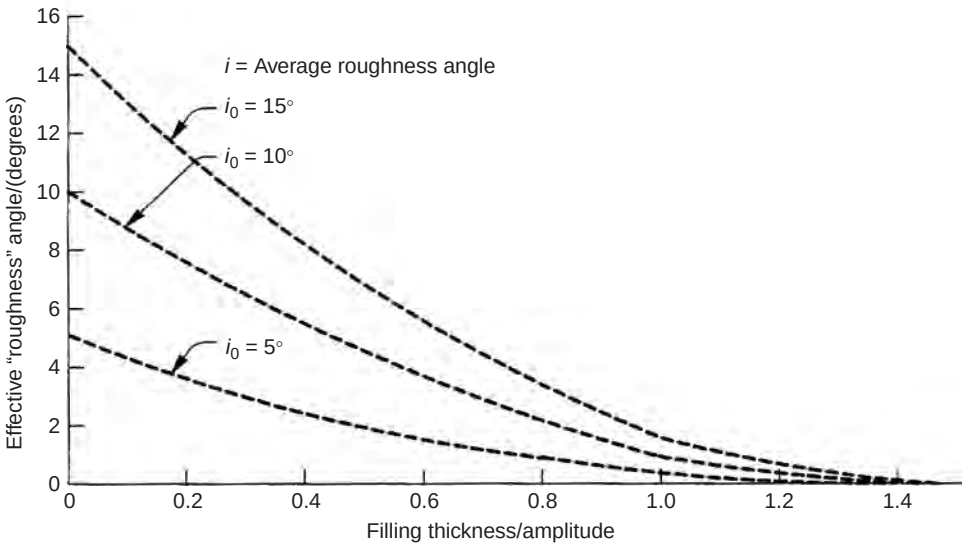


Figure 16.8 The effective roughness angle (i) for seams (infilled discontinuities) (BC Hydro, 1995, based on Ladanyi and Archambault 1977).

joints, bedding and other defects are not oriented close to this failure surface, a Hoek-Brown type failure criterion may be adopted. BC Hydro (1995) recommends the use of the Hoek *et al.* (1995) approach:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left[\frac{m_b\sigma'_3}{\sigma_{ci}} + s\right]^a \tag{16.9}$$

where m_b = a constant based on rock type; s, a = constants dependent upon characteristics of the rock mass; σ_{ci} = uniaxial compressive strength of the intact rock pieces; σ'_1, σ'_3 = effective principal stresses.

The constants m_b, s and a can be estimated using Hoek *et al.* (1995) which relates these constants to the rock mass structure and discontinuity surface conditions.

Douglas (2003) reviewed the basis upon which the Hoek-Brown criteria determined, and has recommended the following:

(a) The basic form of the shear strength equation remains unchanged from the Hoek-Brown criterion.

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(\frac{m\sigma'_3}{\sigma_{ci}} + s_b\right)^{\alpha} \tag{16.10}$$

(b) For intact rock $m = m_i$ and $\alpha = \alpha_i$. These should preferably be measured from triaxial tests on intact rock samples. Alternatively an approximation can be made using the uniaxial compressive strength σ_{ci} and tensile strength σ_{ti} of the intact rock and the equations below.

$$m_i = \left|\frac{\sigma_{ci}}{\sigma_{ti}}\right| \tag{16.11}$$

$$\alpha_i = 0.4 + \frac{1.2}{1 + \exp\left(\frac{m_i}{7}\right)} \tag{16.12}$$

The estimation of m_b, α_b and s_b can be made using the following equations:

$$m_b = \text{minimum of} \begin{cases} m_i \frac{GSI}{100} \\ 2.5 \end{cases} \tag{16.13}$$

$$\alpha_b = \alpha_i + (0.9 - \alpha_i)\exp\left(\frac{75 - 30m_b}{m_i}\right) \tag{16.14}$$

$$s_b = \text{minimum of} \begin{cases} \exp\left(\frac{GSI-85}{15}\right) \\ 1 \end{cases} \tag{16.15}$$

where GSI = Geological Strength Index, exp. = exponential function, e^x.

Douglas (2003) indicates that the equations below from Hoek *et al.* (2002) can be used to estimate the cohesion, c', and friction angle, ϕ', of the rock mass by developing a σ'_n versus τ plot for the relevant stress conditions and rock properties using Equations 16.16, 16.17 and 16.18.

$$\sigma'_n = \frac{\sigma'_1 + \sigma'_3}{2} - \frac{\sigma'_1 - \sigma'_3}{2} \cdot \frac{d\sigma'_1/d\sigma'_3 - 1}{d\sigma'_1/d\sigma'_3 + 1} \tag{16.16}$$

$$\tau = (\sigma'_1 - \sigma'_3)\frac{\sqrt{d\sigma'_1/d\sigma'_3}}{d\sigma'_1/d\sigma'_3 + 1} \tag{16.17}$$

$$d\sigma'_1/d\sigma'_3 = 1 + \alpha m_b(m_b\sigma'_3/\sigma_{ci} + s)^{(\alpha-1)} \tag{16.18}$$

where $d\sigma'_1/d\sigma'_3$ is the slope of the σ'_1 versus σ'_3 plot.

These equations replace those in Balmer (1952), which Douglas (2003) indicates were incorrect.

It is emphasised again that it is quite unusual to have dam foundations which do not have persistent defects which control stability. Unfortunately many dam engineers do not appreciate this and will use the Hoek *et al.* approach for all types of geological environments. This is often done using computer programs such as RocLab which is attractive to some because it allows them to take data from cored boreholes, put them into a spread-sheet and get rock strengths with the push of a button on a computer.

In many if not most cases, that strength will be far higher than the actual strength and dams designed or checked using such strengths may be unsafe.

16.3.3 Tensile strength of rock foundations

Within the scale of most concrete dams, there are discontinuities in the foundation rock mass which mean that the tensile strength is zero, or so close to zero that it should be ignored.

There has in the past been an over-emphasis on the tensile strength of the concrete – foundation contact. This is virtually irrelevant, because the rock beneath the contact has zero tensile strength, so even if the contact has some strength, tensile failure can occur in the foundation immediately below the concrete. In any case even for massive rock often if careful exploratory drilling is done through an existing gravity dam into the foundations, a stained, near-horizontal defect can be found; such a finding would immediately raise doubts about assigning tensile strength to the actual dam/foundation interface zone even for that case.

16.3.4 Compressibility of jointed rock foundation

The Young's modulus of deformation for a jointed rock mass can be estimated from the Geological Strength Index (GSI) using the method proposed by Hoek *et al.* (1998) and Hoek and Brown (1997), using the equation:

$$E_d = \frac{\sqrt{\sigma_{ci}}}{10} 10^{\left(\frac{GSI-10}{40}\right)} \tag{16.19}$$

Douglas (2003) collated a number of case studies from the literature where the rock mass modulus was measured by plate load, pressure meter, dilatometer, flat jack and jointed block tests and the GSI could be estimated. The results from the case studies, together with data presented by Bieniawski (1978) and Serafim and Pereira (1983) are shown in Figure 16.9. The error bars acknowledge the variation of E_d in the test results.

The uncertainty in GSI was due both to the variability of the rock masses and also to the conversion from published RMR values to GSI. The compressive strength of the rock substance lay between 10 MPa and 100 MPa, and the corresponding plots for Equation 16.19 are shown on Figure 16.9. It can be seen there is a large scatter in the data and there is a tendency towards over-estimation of the modulus particularly for lower strength rocks. This needs to be considered when using the equation.

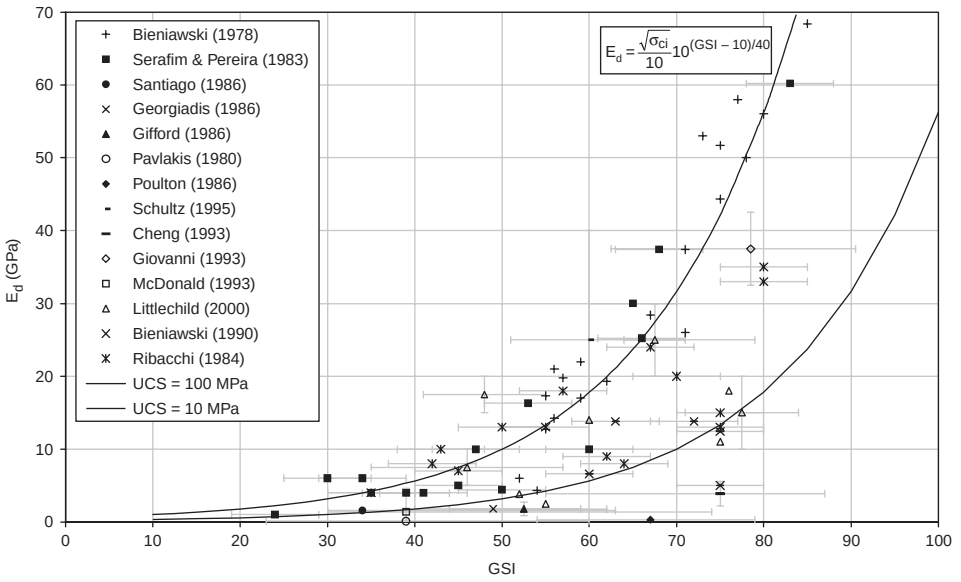


Figure 16.9 E_d versus GSI case study data and Hoek *et al.* (1998) equation for $\sigma_{ci} \geq 100$ MPa and $\sigma_{ci} = 10$ MPa (Douglas, 2003).

16.3.5 Ultimate bearing capacity of rock foundations

The bearing capacity of almost all rock masses is controlled by deformations, not by ultimate strength. This is as for other frictional materials such as sand and gravel. The deformations should be estimated based on moduli determined as described in Section 16.3.4. Failure along kinematically viable failure surfaces in the foundation should be covered in stability analyses.

16.4 STRENGTH OF THE CONCRETE IN THE DAM

16.4.1 What is recommended in guidelines

The tensile and shear strength values of the concrete in gravity dams are sometimes taken from guidelines. Table 16.5 summarizes the recommendations from ANCOLD (2013), CDSA (1999) and FERC (2002).

It should be noted that these recommended values are used only where there is no tensile or shear testing available.

16.4.2 Measured concrete strengths from some USA dams

16.4.2.1 *Background to the data*

Concrete strength test data for cores taken from 23 dams reported in EPR1 (1992) and from test reports on 16 dams from USBR was analysed at UNSW (Khabbaz and Fell,

Table 16.5 Summary of recommended tensile and shear strengths of concrete in dams.

	Guideline and Recommended Value's (4)		
Property	*ANCOLD (2013)*	*CDSA (1999) (1) (2)*	*FERC (2002)*
Static ultimate tensile strength	Strength exceeded by 80% of direct tensile tests. Use zero tensile strength if no tests or if in doubt about lift surfaces	$0.1f'_c$ concrete $0.05f'_c$ lift joints	$0.14(f'_c)^{2/3}$ (3)
Shear strength	0 for unbonded lift joints,	$0.17\sqrt{f'_c}$ concrete	0(5)
– Peak c'	0.95 MPa for bonded lift joints	$0.085\sqrt{f'_c}$ lift joints	
– peak effective friction angle ϕ'	Unbonded lift joints, 45° Bonded lift joints, 57°	55° concrete 45° lift joints	55°(5)

Notes: (1) (a) f'_c = characteristic compressive strength of the concrete in MPa, usually at age 90 days for new dams
(b) Values apply to "normal concrete with well prepared construction joints". Lower values apply for concrete of uncertain quality.
(2) Assumes good quality concrete and lift joints.
(3) Assumes intact concrete
(4) Tensile and compressive strength and cohesion in MPa.
(5) Assumes pre-cracked concrete.

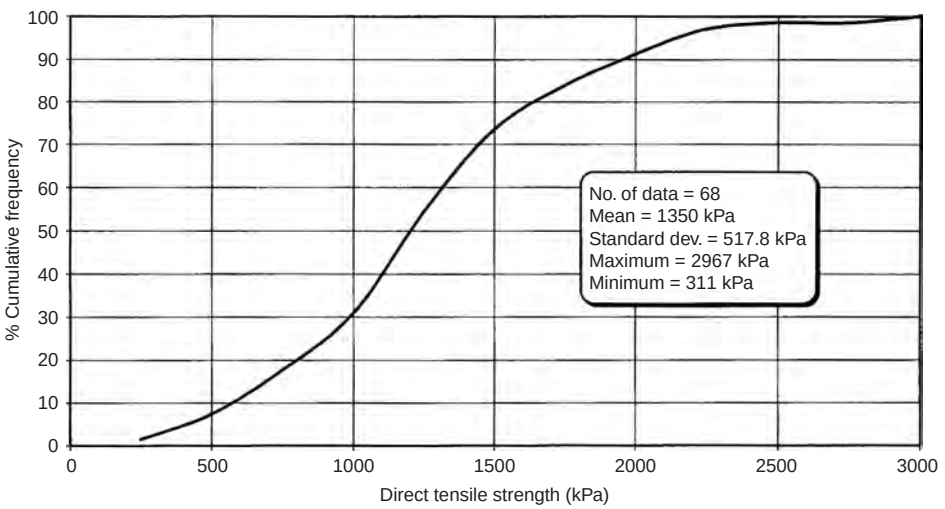


Figure 16.10 Direct tensile strength of concrete with lift joints – some USA dams built after 1940 (Khabbaz and Fell, 1999).

1999). The dams were all constructed by large, expert USA engineering organizations, were in good condition, not showing signs of cracking, weakening of lift joints or serious aggregate – alkali effects.

Three dams gave 'outlier' data because they were affected by alkali aggregate reaction or poor cement quality and these were excluded from the plots in Figures 16.10 to 16.14.

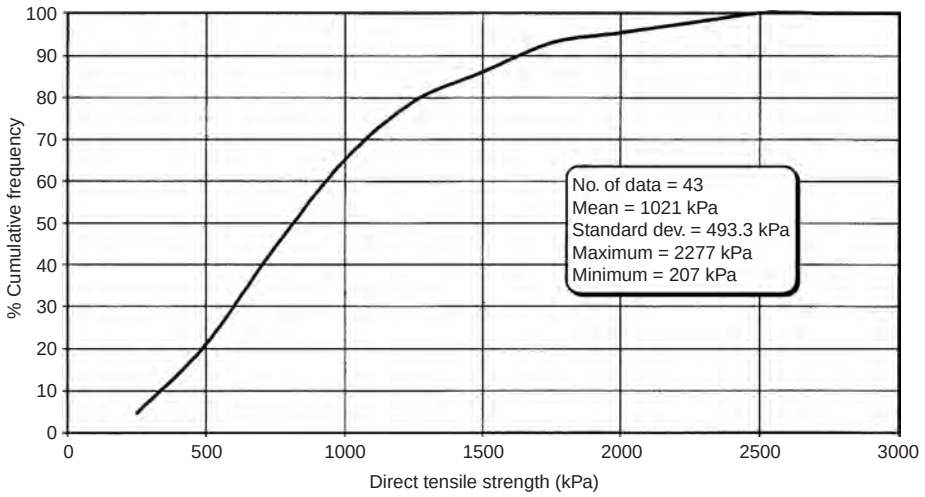

% Cumulative frequency
Direct tensile strength (kPa)
No. of data = 43
Mean = 1021 kPa
Standard dev. = 493.3 kPa
Maximum = 2277 kPa
Minimum = 207 kPa

Figure 16.11 Direct tensile strength of concrete with lift joints – some USA dams built before 1940 (Khabbaz and Fell, 1999).

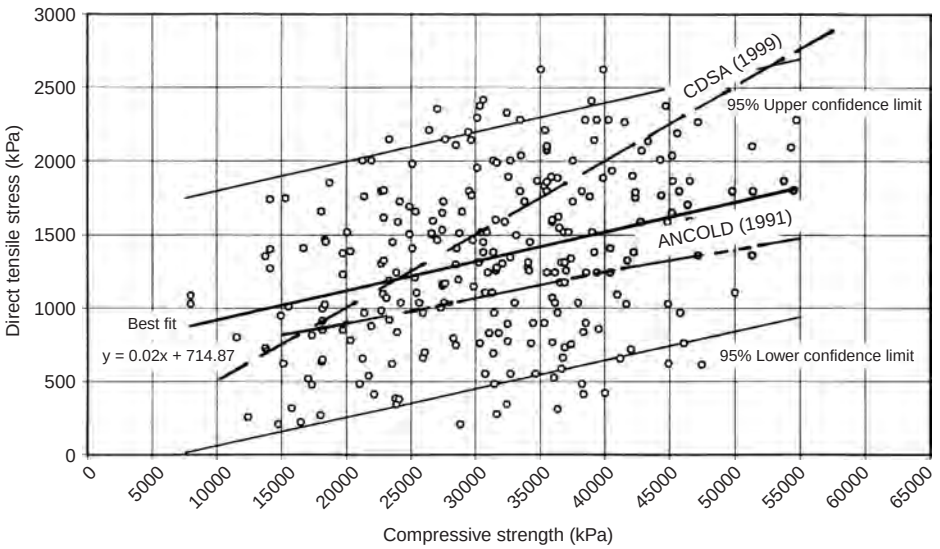


Figure 16.12 Direct tensile strength versus compressive strength – all USA dams data analysed together (Khabbaz and Fell, 1999).

16.4.2.2 *Tensile strength of concrete and lift joints*

Figures 16.10, 16.11 and 16.12 show the results of direct tensile testing on 150 mm or larger diameter cores taken from the dams which had lift joints. Failure often occurred in the concrete, not on the lift joint.

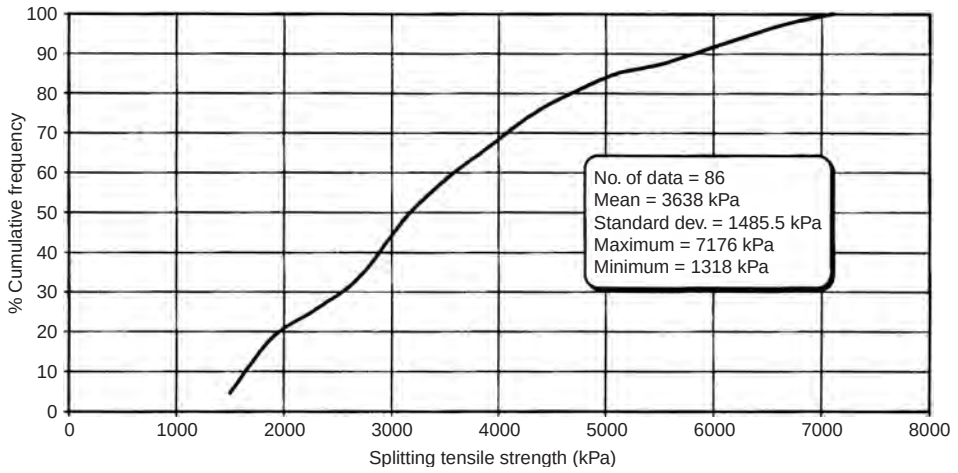

% Cumulative frequency
Splitting tensile strength (kPa)
No. of data = 86
Mean = 3638 kPa
Standard dev. = 1485.5 kPa
Maximum = 7176 kPa
Minimum = 1318 kPa

Figure 16.13 Splitting tensile strength of concrete without lift joints – all data (Khabbaz and Fell, 1999).

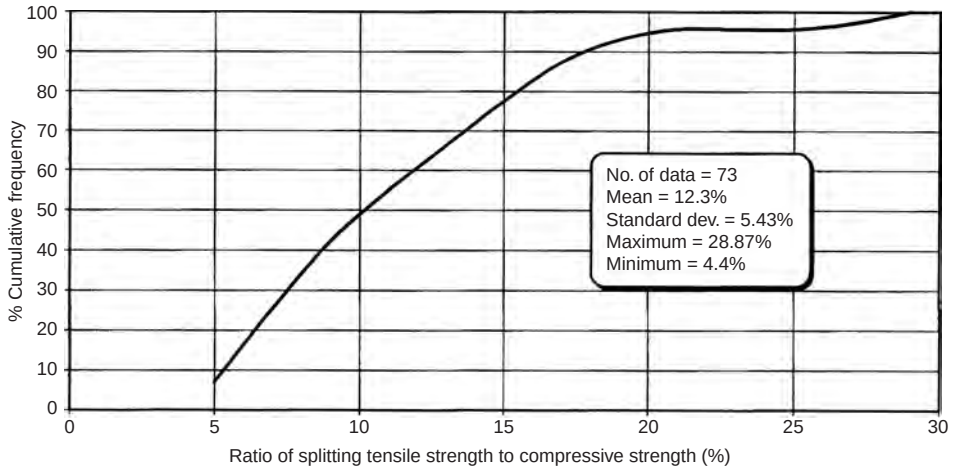
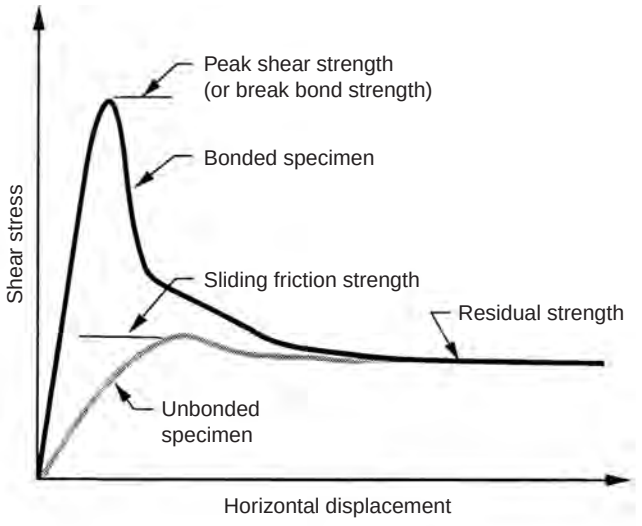

% Cumulative frequency
Ratio of splitting tensile strength to compressive strength (%)
No. of data = 73
Mean = 12.3%
Standard dev. = 5.43%
Maximum = 28.87%
Minimum = 4.4%

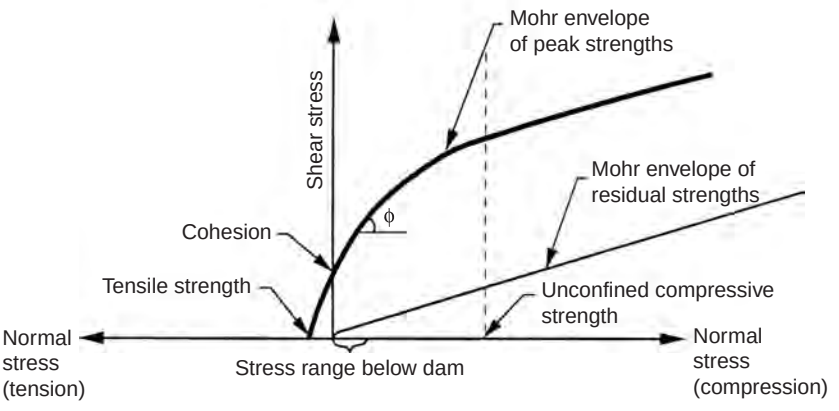
Figure 16.14 Ratio of splitting tensile strength versus compressive strength – of concrete without lift joints – all USA dams data (Khabbaz and Fell, 1999).

Also shown on Figure 16.12 are the ANCOLD (1991) and CDSA (1999) relationships between tensile strength and compressive strength. ANCOLD (1991) had tensile strength $= 0.2\sqrt{f'_c}$. It can be seen that much of the data is lower than the guideline values and there is a poor correlation between tensile and compressive strengths.

Figures 16.14 and 16.15 show results of tests using the indirect tensile test (or splitting test). The strengths are much higher than the direct tensile tests. EPRI (1992) attributes the higher strengths (from the splitting test) to the testing of smaller specimens which tend to fail through the aggregates which have a higher tensile strength



(a) Shear stress – displacement curve



(b) Shear stress – normal stress
curve for intact sample

Figure 16.15 Shear strength terminology (reproduced from EPRI, 1992).

than the concrete mortar. They suggest that the splitting test is satisfactory if the core diameter is 450–900 mm. For small core the direct tensile test is more reliable. USBR (2006) suggests that a factor of two should be applied to the splitting strength to get the direct tensile strength of concrete. This ratio would appear to be reasonable, based on the data in EPRI (1992), provided that the diameter of the splitting test specimens is at least three times the maximum aggregate size in the concrete.

It seems likely the guideline recommendations in Table 16.5 are developed from structural concrete codes, with the strengths usually obtained by splitting tests on small test specimens.

For dams, the direct tensile test results are more realistic, and it would seem unwise to rely on the guideline values. It would be better to do direct tensile tests on core from the dam, or make some assessment of likely strengths from Figures 16.10, 16.11, or 16.12.

It is emphasised that the tensile strength should be assumed to be zero, if

- There are cracks in the concrete, e.g. on lift joints or caused by temperature stresses.
- There are extensive effects of alkali aggregate activity. The data given in EPRI (1992) shows clearly just how much the tensile strength can be reduced by the presence of AAR.
- There is poor quality concrete or
- If there is no reliable data on the condition or strength of the lift joints from construction and cores from the dam.

16.4.2.3 *Shear strength of concrete*

The terminology used by EPRI (1992) and USBR in their testing of concrete in direct shear is shown in Figure 16.15. Samples are described as bonded if they are intact and unbonded if the sample is broken along the plane of weakness being tested.

The peak strength is typically reached at small displacements (0.25–1.25 mm). The residual strength is reached at large displacements but for practical purposes it has been taken as the lowest consistent strength at displacements between 2.5 mm and 12 mm. The sliding friction strength is the peak strength on an unbonded specimen.

The shear strength can be represented in Mohr-Coulomb terms as:

$$S = c' + \sigma' \tan \phi' \tag{16.20}$$

where S = shear strength; c' = effective cohesion; σ' = effective normal stress; ϕ' = effective friction angle.

The strength envelope is actually curved and at low normal stresses the friction angle is high, but approaches the residual value at high normal stresses. This non-linearity needs to be considered in design.

EPRI (1992) report the following results from testing of concrete lift joints:

Peak strength:	best fit	$c' = 2.1$ MPa, $\phi' = 57°$
	90% stronger than	$c' = 0.95$ MPa, $\phi' = 57°$
Sliding friction strength:	best fit	$c' = 0.5$ MPa, $\phi' = 49°$
	90% stronger than	$c' = 0$, $\phi' = 48°$

Figures 16.16 to 16.19 show test data. It can be seen, that as suggested by EPRI (1992), it is best to use a bi-linear relationship to model the sliding friction strength.

There is little difference between sliding friction shear strength and residual strength – only 2° to 3° in the data available. The lower bound of the sliding friction strength data is $c' = 0$, $\phi' = 34°$. Many of the data near the lower bound are from Stewart Mountain Dams, which displayed alkali-aggregate reaction, with silica-gel noted on the surfaces of open joints. If these data are excluded the lower bound becomes $c' = 0$, $\phi' = 38°$.

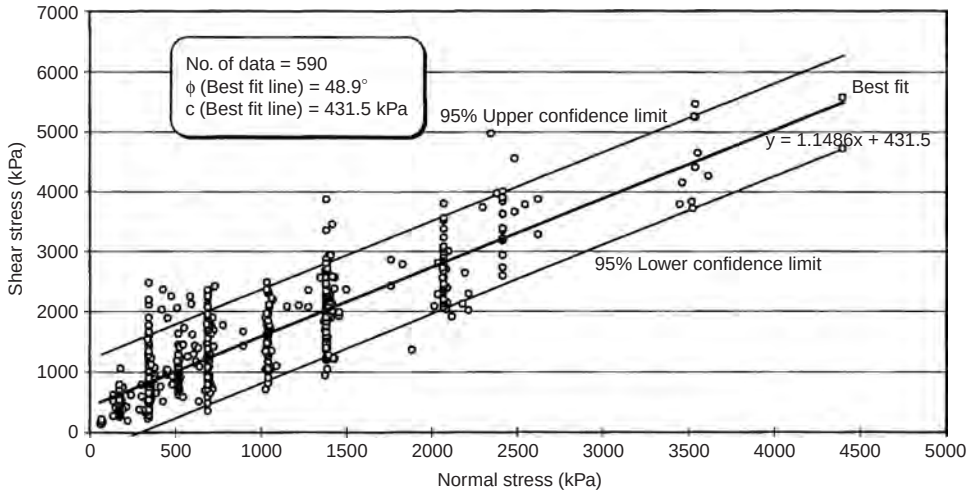


Figure 16.16 Sliding friction shear strength of concrete with lift joints – all USA dams data (Khabbaz and Fell, 1999).

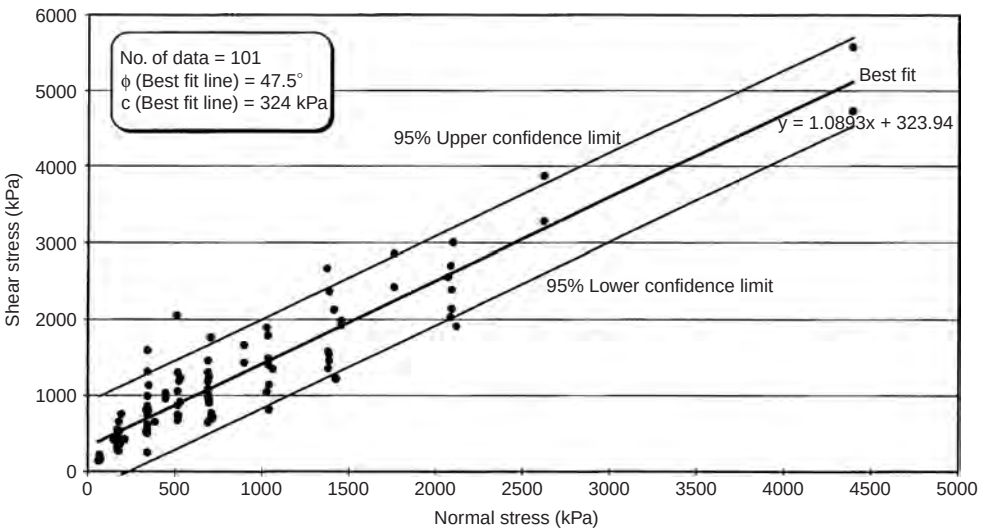


Figure 16.17 Sliding friction shear strengths of concrete with lift joints – USA dams built after 1940 (Khabbaz and Fell, 1999).

The recommended strengths from guidelines given in Table 16.5 are on the conservative side if there has been no displacement on the lift surfaces. However only small differential movements, e.g. from thermal expansion and contraction, would take the strength towards the sliding friction strength. It is suggested that the EPRI (1992) data be used along with Figures 16.16 to 16.19 to estimate the strength. Where it is critical, or there are weak lift joints, laboratory tests should be carried out.

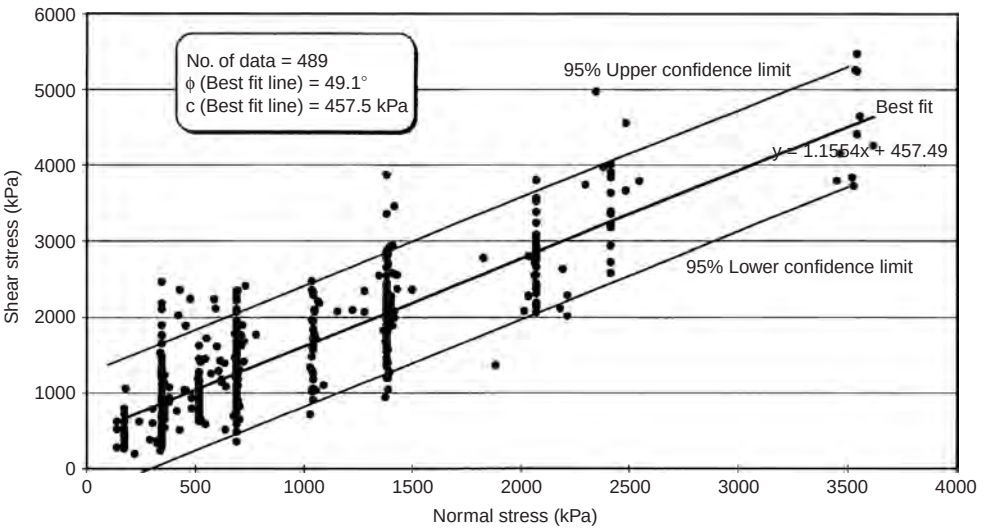


Figure 16.18 Sliding friction shear strengths of concrete with lift joints – USA dams built before 1940 (Khabbaz and Fell, 1999).

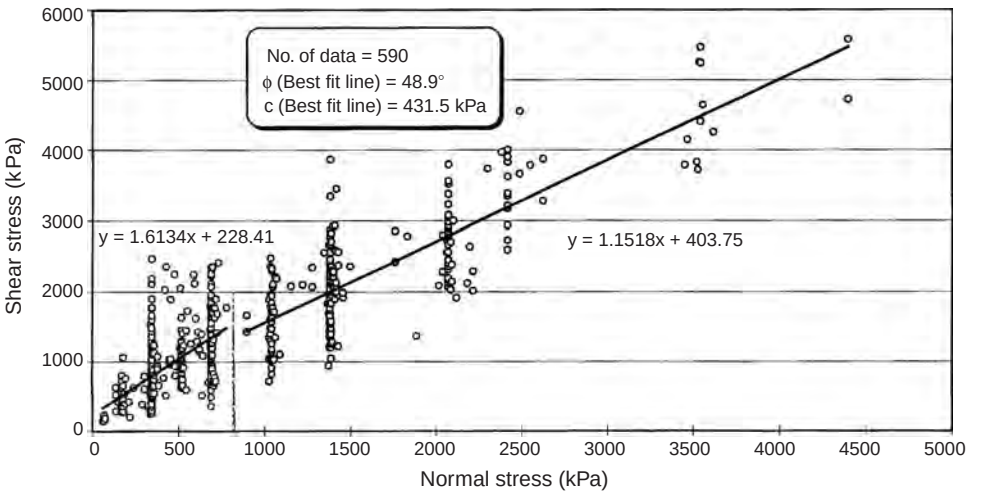


Figure 16.19 Sliding friction shear strength of concrete with lift joints – all USA dams data, split at 750 kPa (Khabbaz and Fell, 1999).

16.5 STRENGTH OF THE CONCRETE – ROCK CONTACT

EPRI (1992) report on direct shear tested on the concrete-rock contact. They indicate that peak friction angles are large, typically 54° to 68° at low stresses.

Khabbaz and Fell (1999) plotted the available data which is shown in Figures 16.20 and 16.21.

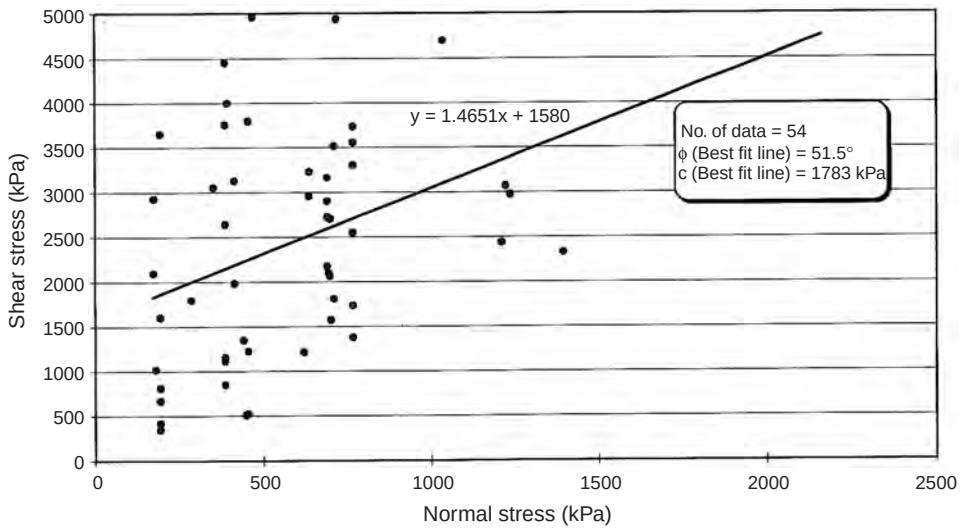


Figure 16.20 Peak shear strength of concrete-foundation contact – all USA dams data (Khabbaz and Fell, 1999).

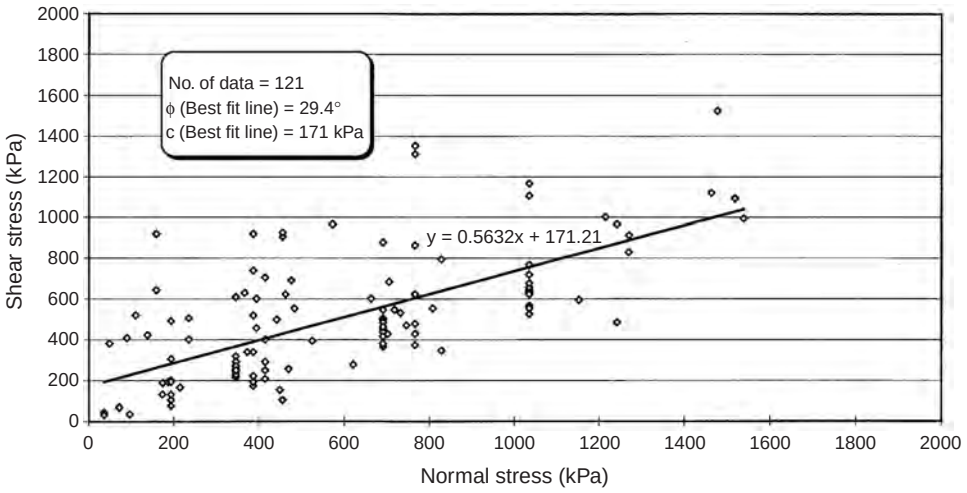


Figure 16.21 Sliding friction shear strength of concrete-foundation contact – all USA dams data (Khabbaz and Fell, 1999).

Provided good construction practice has been followed, there is no reason the contact strength should control. Rather it should be the lower of the concrete strength on lift joints or the rock foundation shear strength. However, some existing dams could have had inadequate excavation and preparation of the foundation surface. Potentially unfavourable conditions that may exist include weathered or blast-damaged rock, consolidation and/or slush grout at the base of the concrete or a deteriorated contact.

In these cases the contact strength may control, and any extensive areas of weak contact material should be sampled and the strength determined in the laboratory.

The tensile strength of the contact should be assumed to be zero – because it is controlled by the tensile strength of the rock mass immediately below the contact.

16.6 UPLIFT IN THE DAM FOUNDATION AND WITHIN THE DAM

16.6.1 What is recommended in guidelines?

For new dams, or for assessing existing dams for which there are no measurements of pore pressures in the rock foundations or on potential failure surfaces within the dam, there are well-established empirical rules for estimating uplift pressures. Figures 16.22(a) and (b) show typically used assumptions, in this case as proposed in FERC (2002).

The pressures depend on whether the drainage gallery is above or below the tailwater level and the efficiency of the drains.

For Figure 16.22(a) theoretical crack does not extend past line of drains:

For H4 > TW

$$H3 = K\left[(HW - TW)\frac{(L - X)}{(L - T)} + TW - H4\right] + H4 \tag{16.21}$$

and for H4 < TW

$$H3 = K\left[(HW - TW)\frac{(L - X)}{(L - T)}\right] + TW \tag{16.22}$$

where K = (1 − E); and E = drain efficiency as a fraction. E = 0 for blocked drain condition and E = 100% for full efficiency.

For Figure 16.22(b) theoretical crack does extend past line of drains:

For H4 > TW

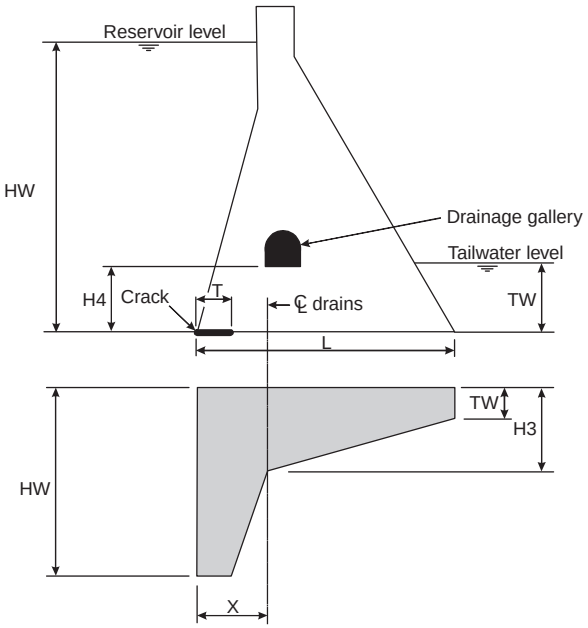
$$H3 = K(HW - H4) + H4 \tag{16.23}$$

and for H4 < TW

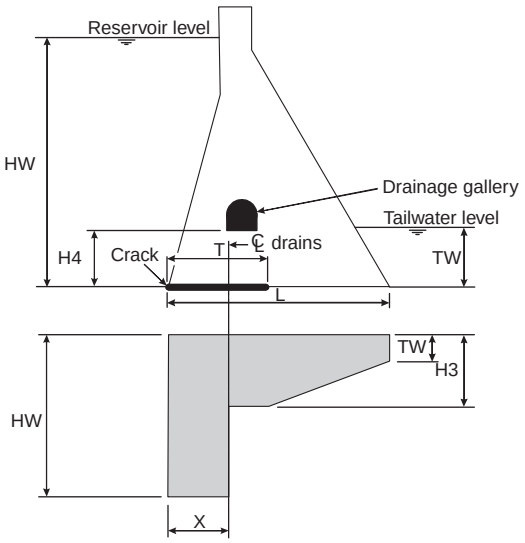
$$H4 = K(HW - TW) + TW \tag{16.24}$$

FERC only advocates the adoption of the uplift distribution shown in Figure 16.22(b) if there is clear evidence that the drains are effective once the crack goes past the line of drains. If such evidence is not available, then, the drains should be ignored as they may be unable to relieve the pressure and the pressure distribution along the base of the dam assumed as for the no-drains case given below.

As discussed in USACE (2000a), the USACE follows a similar approach, except that it assumes that the drains will not be effective once the crack from the heel theoretically passes the line of drains. The USACE may vary this latter requirement, if the



(a) For no-compression zone not extending to the drain



(b) No-compression zone extending downstream of the drains

Figure 16.22 Uplift assumptions for concrete gravity dams (adapted from FERC, 2002).

costs of the suggested remedial works are found to be very large. The USBR is even more conservative in that it requires evidence of the effectiveness of the drains before the drainage system can be allowed for when a crack theoretically develops from the heel of the dam (see discussions in USACE (2000).

For the no drains case (or drains blocked), the usual assumption is for 100% of the reservoir head at the upstream face (heel) to the root of the theoretical "crack", if one has been shown by analysis to exist, to 100% of the tailwater head at the downstream face (toe) as a straight line function.

A common assumption (e.g. ANCOLD, 1991, 2013; BC Hydro, 1995) for the case with a well constructed and maintained borehole drain system extending deep into the foundation below any potential failure surface, is for the drain efficiency (E) to be 67%; (or K = 0.33) for the normal operating and flood cases.

FERC (2002) do not suggest this, recommending that the efficiency must be measured by piezometers in the dam foundation. They point out that the measured drain efficiency must only be considered valid for the reservoir water levels for which measurements are taken, and warn that linear extrapolation to higher levels may not be valid. This is also emphasized by EPRI (1992) who give a number of examples of non-linear uplift response to reservoir level. The authors agree with this approach in principle, but we do point out that, unless there are remote means to measure the pore pressure changes during a flood, it would probably be very unusual for engineers to satisfy FERC's direct measurement requirement of pore pressures to confirm the drain efficiency factor under flood loading conditions.

It should be noted that most engineers will use a lower tailwater level to compensate for any drawdown caused by spillway discharge when calculating the load on the dam due to tailwater. For the calculation of uplift pressures on the base of the dam, the full tailwater head must be used.

Figure 16.23 shows an example of the uplift distribution on a discontinuity in the dam foundation.

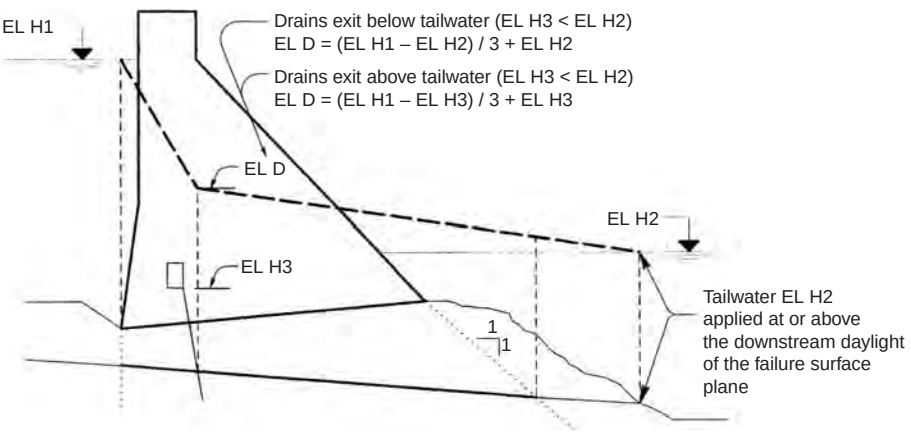

EL H1
Drains exit below tailwater (EL H3 < EL H2)
EL D = (EL H1 – EL H2) / 3 + EL H2
Drains exit above tailwater (EL H3 < EL H2)
EL D = (EL H1 – EL H3) / 3 + EL H3
EL D
EL H2
EL H3
1
1
Tailwater EL H2
applied at or above
the downstream daylight
of the failure surface
plane

Figure 16.23 Example of uplift distribution on a discontinuity in the dam foundation (BC Hydro, 1995).

Note that the tailwater level pressure applies where the potential failure surface emerges into the tailwater, and the pressure at the drains is where they intersect the potential failure surface.

16.6.2 Some additional information on uplift pressures

16.6.2.1 *Effects of geological features and deformations on foundation uplift pressures*

EPRI (1992) present a summary of the influence of geological features and deformations on foundation uplift pressures. They demonstrate that the assumption of linear uplift pressure distributions from the reservoir to tailwater for the no drains case, and from the drains to the tailwater in the drained case, are potentially not conservative because the discontinuities in the rock foundation can be narrower at the downstream than upstream because of the stresses and deformations induced by the dam and the reservoir load on the dam, and by geological features which limit the flow of water.

Figure 16.24 shows the effect of block sliding and rotation in the foundation, Figure 16.25 the effect of varying joint apertures and Figure 16.26 the effect of a low permeability shear zone.

This emphasizes the need for careful consideration of the foundation geology in designing new dams, or assessing existing dams which do not have piezometers in the foundation. However there is no substitute for actual measurement of the pressures, with a carefully designed and well-monitored system of piezometers.

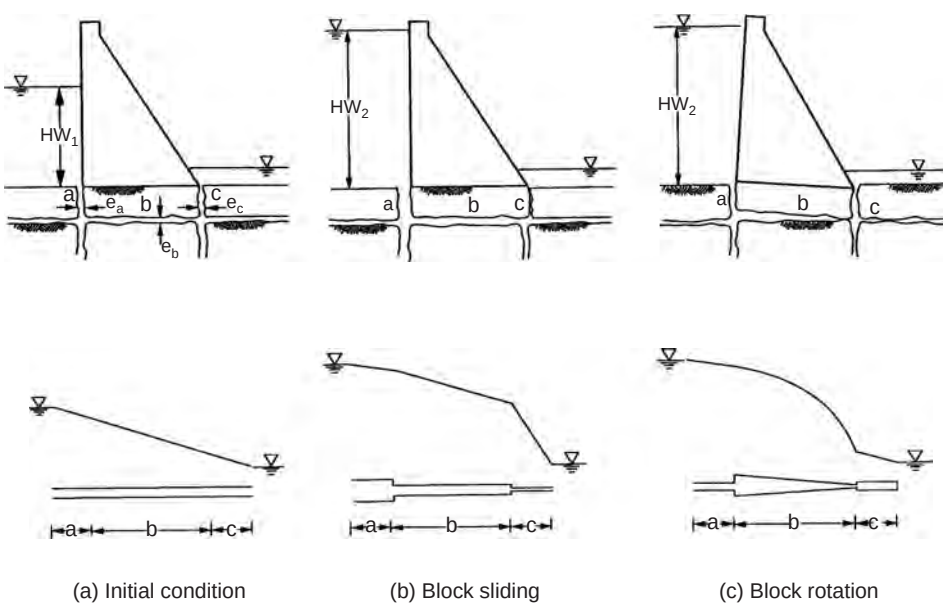


Figure 16.24 Effect of block sliding and rotation on joint apertures uplift pressure. (a) Initial condition, parallel sided joint; (b) Block sliding, joint c width reduced, joint a width increased; (c) Block rotation, joint b narrows at downstream end (adapted from EPRI, 1992).

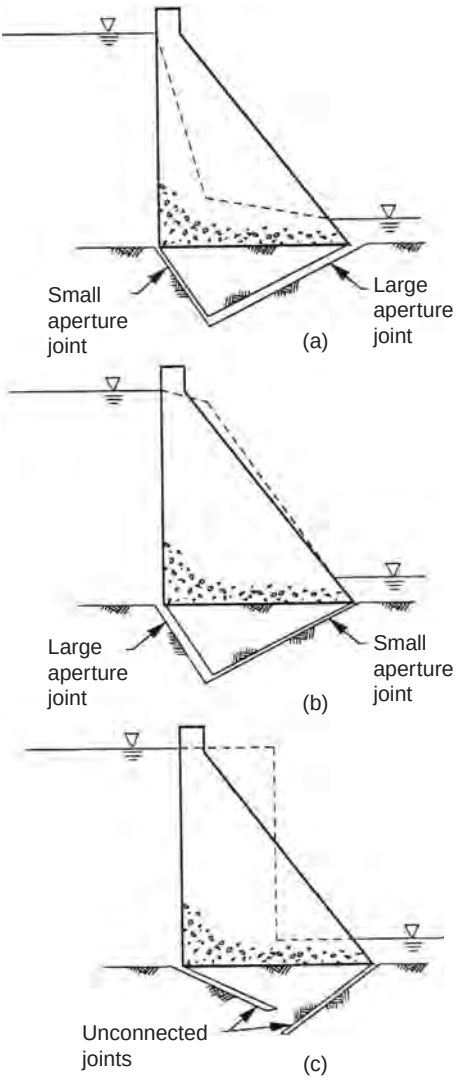


Figure 16.25 Effect of joint apertures and continuity on uplift pressure (adapted from EPRI, 1992).

16.6.2.2 *Analysis of EPRI (1992) uplift data*

An analysis by Khabbaz and Fell (1998) of uplift data recorded in EPRI (1992), gave the results shown in Figure 16.27. In this figure the uplift coefficient α is the ratio of the measured pressure above tailwater level to the difference between reservoir level and tailwater level. It is equivalent to K in Equations 16.20 to 16.23. Data from 24, mostly USA, dams was available and represent the maximum pressures recorded across each dam. X/L is the ratio of the distance from the line of drains to the measuring point

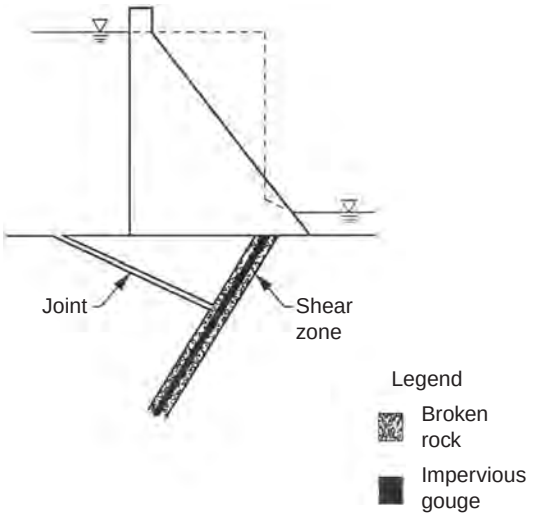


Figure 16.26 Effect of low permeability shear zone on uplift pressure (adapted from EPRI, 1992).

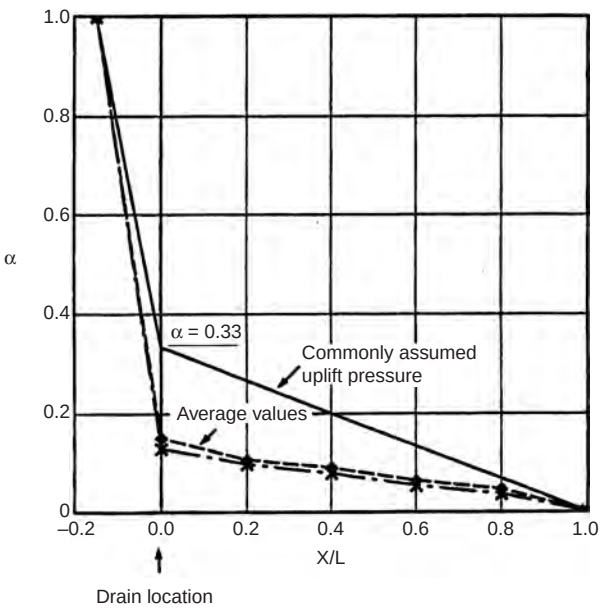


Figure 16.27 Observed maximum uplift pressures in some USA dams (Khabbaz and Fell, 1998).

compared to the distance from the line of drains to the downstream toe of the dam. Table 16.6 summarises the data.

This information shows that the 'standard method' assumption of $\alpha = 0.33$ at the drains is usually conservative. The highest value recorded was 0.47 for piezometers in

Table 16.6 Maximum uplift coefficients observed in some USA dams (Khabbaz and Fell, 1998).

X/L	−0.15	0*	0.2	0.4	0.6	0.8	1
Mean α	1	0.150	0.105	0.089	0.062	0.045	0
Standard deviation for α	–	0.123	0.082	0.087	0.076	0.06	–

*Location of drain holes.

one block of a dam where the drains were somewhat blocked, and the next highest, 0.33 for a dam with drains spaced at about 11 m centres rather than the normal 2 metres. However analysis of some Australian dams showed some had areas with maximum uplifts greater than the 'standard method'.

Analysis of the data to assess the effect of depth of drains, height of dam and other variables showed few clear trends. There did seem to be a tendency for lower uplift pressures for high dams, which may reflect greater care in the construction of the drains or grout curtain upstream of the drains, or that the weight of the dam closes discontinuities in the foundations.

16.6.2.3 *Design of drains*

Foundation drainage is provided by the provision of boreholes drilled into the dam and the foundation from the drainage gallery. ANCOLD (2013) recommend the following to achieve the uplift coefficient of 0.33 which is commonly assumed:

(a) The line of drains should be located at a distance downstream from the upstream face of between 5% and 15% of the maximum reservoir head at the dam.
(b) The lateral spacing of the drains within the dam should not exceed 10% of the maximum reservoir head at the dam, and preferably should not exceed 3 m. However, in the body of the dam drains should not be so closely spaced as to encourage cracking between them.
(c) The lateral spacing of drains within the foundation should not exceed 3 m.
(d) The drains within the body of the dam should be drilled or formed at least 75 mm in diameter and preferably 100–150 mm diameter.
(e) The drains drilled into the foundation from a lower gallery should extend well below any potential failure surface, and be a minimum of 75 mm diameter and preferably 100 mm diameter.
(f) The line of drains should be oriented to intersect rock defects. Where they do not (e.g. vertical holes where there are significant vertical defects) the drains are unlikely to be fully effective as vertical defects will not be intersected by the drains.
(g) The line of drains should extend the full length of the dam unless other design precautions are taken for those sections not covered by drains.
(h) Monitoring of the drainage system should be done if the design uplift relies on reduction at the drains. Monitoring typically consists of assessing that the drains are still running, probing to determine effective depth and reading piezometers.
(i) The drains should be cleaned regularly (rodding, reaming or re-drilling/water blasting) to remove any infill or calcite present.

The authors emphasise that it is essential that the drains penetrate well below the level of any potential failure surface and that they are oriented to intersect the defect system. Hence for horizontally bedded rock with a sub-vertical joint system, it is essential that the holes be drilled inclined to the vertical to intersect the joints. The depth should be determined from the geology of the foundation and the geometry of the dam, but it is unlikely that drains, which do not penetrate at least 4 m below a potential failure surface, would be effective because they have to drain the jointed rock mass below the potential failure surface to relive pressures on the failure surface. It would be wiser for them to be at least 6 m below the potential failure surface.

The drains should be constructed after the dam is constructed and the grouting of the grout curtain, (which is usually drilled and grouted from the drainage gallery and located upstream of the drain holes), is completed. At this time the stresses and deformations induced by the dam are present and the grouting cannot block the drainage system.

As pointed out by FERC (2002) the grout curtain may retard flows initially but the degree of uplift reduction is difficult to predict and may reduce with time as the cement in the grout is leached. Grouting alone (without drains) should not be considered sufficient justification to assume a reduced uplift.

Vertical drainage holes in the concrete of a gravity dam inevitably require a gallery to work. The question of the effectiveness of these drains is one that is often asked. Under normal operating conditions when in most dams there is compressive effective stress over the whole section, any seepage water that did reach the drains through the natural porosity of the concrete would cause the outlet at the drain to be sealed with calcium carbonate. The reaction here involves the seepage water picking up excess lime (CaO) from the concrete's matrix to form $Ca(OH)_2$ which combines with CO_2 in the air at the exit to give $CaCO_3$. If the seepage's exit velocity is small enough, the $CaCO_3$ comes out of solution to create the seal. The drains therefore would be ineffective in reducing internal pore pressures. The same outcome would occur if a small crack extended to the drain, as the exit velocity of the seepage would be very small. Only for substantial cracks would the seepage water discharge into the drains continuously. Even then, over time $CaCO_3$ would probably come out of solution as the water trickled down the drain hole, a situation that can be seen in many concrete dams. Unless precautions are taken, the authors believe that drain within the dam wall should be ignored in the design of a new dam or assessment of an old dam, unless calculation show that a crack or cracks would extend to the drains and only then for short-term loading conditions.

Over recent years, one approach to try to keep the drains within the concrete effective is to fit an air-trap at the outlet end of each drain in the gallery and to make sure the top is adequately sealed (if at all possible with a removable plug so that later inspection and maintenance can be done from the top of the hole). The object is to exclude CO_2 from entering the hole or at least to minimise the chance of that happening. Once the CO_2 in the hole is used up, the argument is that water should be able to seep freely into the hole and thence through the air-trap. Precipitation of $CaCO_3$ should be kept within the trap or where the water ends up on the gallery's floor drain. This method was used in several of the old gravity dams owned by the Sydney

Catchment Authority and in the recently constructed Enlarged Cotter Dam, an 85 m high RCC dam near Canberra in the ACT.

It will be understood that the cost of cleaning these drains can be significant as reaming is often needed and that operation can be very difficult unless the drains were formed generally as straight lines or were drilled after the concrete was placed. If the drains in the dam wall do play a key role in the dam's stability, then this cleaning operation should extend to them as well as to the foundation drains. In new dams, therefore, care must be taken to detail all drains so that cleaning can be done without the expenditure of a lot of money. Without a regular cleaning program in place, full reliance on the drains must be viewed with some doubt.

It is therefore suggested that within the dam's concrete, uplift across any section should be assumed as varying linearly between the pressure at the upstream face and zero or tailwater level at the downstream face (drain ignored in this case). This is consistent with modern guidelines. For a crack-free dam, such an assumption may be conservative. Some expect the pressure to fall rapidly within a short distance of the upstream face of the dam. US Corps of Engineers (1995) suggest that a 50% reduction in pressure across the section would be acceptable to account for the likely sudden pressure drop at the upstream face, but the warning is given that such an assumption would apply to crack-free concrete. Given it is difficult to prove there are no cracks, the preferred approach is to assume linear pressure as described above.

The appearance of $CaCO_3$ on the downstream face of a dam, at, say, a construction joint, could be interpreted as a crack with full head applied up to the blockage, in this case, the downstream face. Even a formed drain in the concrete might not help as there could be a similar $CaCO_3$ seal here. This particular condition seems to have been ignored in most codes. Unless the 'leak' is wide spread, in most dams the localised effect would be countered by the crack not propagating. The authors believe that a conservative attitude to pore pressures within the dam wall would cope with any such localised pressure development.

$CaCO_3$ formation can also occur in dams within the foundations, below the water surface in the drains. The reaction here involves CO_2 being released by microbes in the ground and H_2CO_3 being formed. This very mild acid then reacts with the excess lime from, say, a grout curtain and $CaCO_3$ results. This phenomenon has been seen at the pipeline inlet structure, a 34.7 m high gravity dam, at the head of the pipelines to Tumut 3 power station in the Snowy Mountains Scheme.

The possible formation of $CaCO_3$, limonite and other products in the foundation drains clearly points to the need to clean these drains regularly. At best, this cleaning would include high pressure water jet cleaning; at worst, the holes may have to be reamed. The high pressure water technique works well in hard rock, but the danger is that if the odd weaker zone (fault, soft seam and the like) is met, a lot of soft material can be quickly washed out. The safest, but by far the more expensive way is to ream the holes. Only the Owner can decide at what intervals the maintenance is needed. As a guide it should be done within the first 10 years of the life of a dam; the results of that exercise would be a guide as to what the future intervals are likely to be. Likewise in an existing dam that is about to be have its drains cleaned for the first time, the results of that operation should be carefully examined and a future program of cleaning worked out.

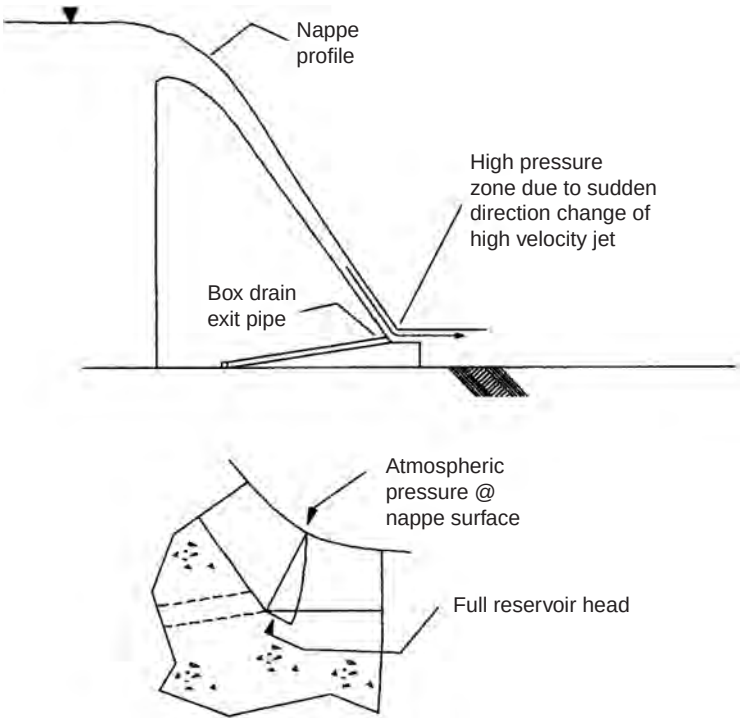


Figure 16.28 Effect of poorly located drain holes on uplift pressures (FERC, 2002).

16.6.2.4 *Hydro-dynamic forces*

Spillway sections of concrete dams are subjected to additional hydro-dynamic forces due to the passage of water over the crest.

At the design discharge capacity, an ogee crest has zero pressure at the crest due to the water. However at discharges higher than the design discharge, there is a negative pressure which adds to the overturning moment and must be accounted for. These effects are shown in Figure 16.1. The negative pressures are most significant in small height structure such as spillway crests leading to a chute spillway.

In a flip bucket or any location where flow is changing direction in a concave upwards manner, the water exerts positive pressures. These may also contribute to overturning if they are downstream of the centre of the base of the dam.

Methods for calculating these pressures are given in FERC (2002), Brand (1999), and USBR (1990).

As pointed out by FERC (2002), and shown in Figure 16.28, if the outlet for a drain is in the flip bucket of a spillway, it may be subject to very high pressures. Such pressures can also be transmitted to the foundation through open joints in the spillway, or poorly positioned drain holes in spillway aprons. These situations should be avoided or, if present in a dam, the drainage system should be amended.

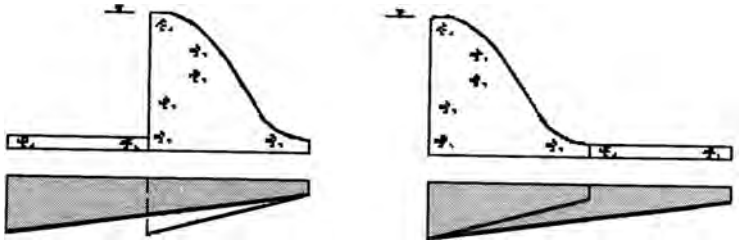


Figure 16.29 Effect of aprons on uplift pressures: (a) Upstream apron reduces uplift pressure, (b) Downstream apron increases uplift pressure (FERC, 2002).

As discussed by FERC (2002), the uplift pressures in the dam foundation will respond rapidly to reservoir levels because in a saturated rigid rock foundation, extremely small volume changes are required to transmit the pressures. Such behaviour has been clearly recorded for many years at Tumut Pond dam (an arch dam). In the absence of measured data to prove otherwise, the uplift should be assumed to vary linearly with changes in reservoir and tailwater levels. However there are examples (EPRI, 1992) of non-linear response giving higher pressures.

16.6.2.5 *Aprons*

Upstream and downstream aprons have the effect of increasing the seepage path under the dam. For an upstream apron properly sealed to prevent leakage, the effect is to reduce the uplift under the dam. The effectiveness of upstream aprons in reducing uplift is compromised if cracks and joints in the apron permit leakage.

Downstream aprons, such as stilling basins or spillway chutes have the effect of increasing uplift under the dam as shown in Figure 16.29.

Many stilling basins or spillway chutes have drains constructed at the concrete-rock contact. These may, if they are well constructed and maintained, reduce the pressure at the interface, but as discussed in Section 16.6.2.6, they will not be effective for potential failure surfaces within the rock foundation.

It should be noted that there should be no drains exposed in the floor of the stilling basin. The potential for transient pressures from the pounding that might happen within the stilling basin during spillway operation to affect the foundation below the basin could result in the basin and the rock being subjected large cyclic loading.

16.6.2.6 *'Contact' or 'box' drains*

Many older, smaller concrete dams and spillways do not have a drainage gallery and drain holes bored into the foundation, but are provided with a gravel filled drains at the contact between the concrete and the rock.

It is the authors' experience that too much reliance has been put on these drains to reduce uplift pressures.

Provided they are well constructed, and can be cleaned or inspected to ensure they are not clogging, contact drains should reduce uplift pressures at the concrete-rock contact. However as pointed out by EPRI (1992) they will not reduce pressures in the rock, because the flow of water in the foundation is controlled by the fracture system. If the critical failure surfaces are in the rock, as they usually will be, there will be little benefit and the effect of the contact drains should be ignored.

16.7 SILT LOAD

Sediment in the reservoir, which deposits against the upstream face of the dam exerts a pressure on the dam and must be allowed for in the analysis. For small structures with a heavy sediment bed load this may be substantial since with time sediment will partly or completely fill the storage up to spillway crest level.

The load applied by the 'silt' should be estimated using Coulomb earth pressure theory, assuming the at-rest, or K_o, condition applies. The sediment is likely to be loose, so in the absence of specific data, properties consistent with loose material should be used.

For preliminary assessments only, the following properties could be used to assess the pressures:

(a) For weirs and small structures in fast flowing rivers and streams where the sediment consists mainly of sand and gravel, adopt a saturated unit weight of 20 kN/m^3, an effective friction angle of 30° and effective cohesion zero.
(b) For structures where sediment is likely to be fine-grained silt and sand, adopt a saturated unit weight of 17 kN/m^3, an effective friction angle of 25°, and effective cohesion zero.

In the calculation of the silt pressure, if silt is submerged, the saturated unit weight of the soil should be reduced by the unit weight of water to determine the buoyant weight.

When assessing dams under earthquake loading, the possibility of liquefaction of the 'silt' should be assessed. If this is likely, the silt loads will be greatly increased because the strength of the soil is reduced and the load will approach a $K_o = 1.0$ condition.

16.8 ICE LOAD

Where the reservoir may freeze, the ice will exert additional forces on the dam. FERC (2002) and US Corps of Engineers (1982) give guidance on how to estimate the forces.

16.9 THE DESIGN AND ANALYSIS OF GRAVITY DAMS FOR EARTHQUAKE LOADING

16.9.1 Introduction

This section deals with the earthquake loading of gravity dams. It concentrates on the more general aspects of the analysis rather than discussing the details of the methods

and the analytical tools. Background data on these aspects, particularly of the finite element methods, can be found in ICOLD (2012), USACE (2007), FERC (2002) and in the manuals produced by companies that have developed the finite element packages. Relevant design guidelines, such as ANCOLD (2013), USACE (2007) and FERC (2000), should be used to define the dam's design earthquakes (SEE and OBE in Chapter 12), permissible stresses and so on. Attention is also drawn to the earlier parts of this chapter for design strength parameters.

In the minds of most dam engineers today, the design and analysis of gravity dams for earthquake loading would mean either the pseudo-static method or the more complex approach of a finite element analysis.

16.9.2 Gravity dams on soil foundations

Generally speaking the foundations will be the critical factor in the design of these dams for earthquake loading. The approach, therefore, to check these dams for earthquake loading would logically be to follow the procedures outlined for embankment dams in Chapter 12. For example, one would use SHAKE to determine the ground shaking accelerogram for the soil deposit and then undertake a Newmark displacement analysis to estimate the permanent displacements along a selected failure surface through the soil foundations. As most of these dams would be low, stiff structures, the dam's response effects could probably be ignored.

As will be clear from the discussion in Chapter 12, the net outcome could very well be significant permanent displacements within the foundation that could, in turn, seriously compromise the dam. For a new dam, the object would be to design the dam such that its static load factors of safety are high enough so that under earthquake loading the resultant permanent displacements would most likely be small. As well, the dam's details would need to cope with potential differential movements between blocks. For an existing dam, unless considerable strengthening works can be justified, an owner may simply have to live with the inherent risk. Finally, for both new and existing dams, great care will be needed to assess the risk of post-earthquake internal erosion and piping through the foundations.

In the case of concrete dam (or spillway structure) being founded on potentially liquefiable foundations measures will have to be taken to overcome the effects of liquefaction such as removal or densification of the liquefiable material.

16.9.3 Gravity dams on rock foundations

16.9.3.1 *General*

The methods of analysis commonly used extend from the simple pseudo-static method first suggested in Westergaard (1935), a method that treats the dam on a rigid body (natural period = zero seconds) to complex finite element methods in which there are one or more non-linear features, usually in the form of elements representing cracks.

In between the two limits of the simple pseudo-static approach and the complex FEM are pseudo-static methods such as the one developed by Fenves and Chopra (1987). This method allows for the use of the first (the natural) period and, if needed, higher periods, to assess the impact of the dams response to the ground shaking. The

US Corps of Engineers developed an approach that starts with this type of method and extends to limited FE analyses (Guthrie, 1996).

16.9.3.2 *The Westergaard pseudo-static method*

This method, which is usually applied in a 2-dimensional analysis, treats the dam as a rigid structure on a rigid foundation. The hydrodynamic effects, which Westergaard determined from harmonic ground motion, are included in the analysis by attaching a 'block of water' to the face of the dam. This block or mass of water, perhaps the first time that the term "added mass" was used, has a shape for a vertical faced dam of:

$$b = \frac{7}{8}\sqrt{hy} \tag{16.25}$$

where b = width of the added mass measured normal to the face horizontal at depth y.
h = height of the dam at the section concerned (generally the height above the reservoir floor, not necessarily the structural height of the dam)

The total inertia force is found by applying the design peak ground acceleration (pga) to the structure. The moments and shear forces at the base and at other sections follow in the normal way for a conventional static load analysis.

FERC (2000) advises that this method should not be used and the authors would agree. It may be reasonable for low pgas but once these pgas are 0.2 g and beyond, the analysis simply proves that the dam would be unsafe, particularly if the engineer has assumed an $f_t = 0$ foundation. This outcome is probably not helped by designers treating the analysis as a conventional static load analysis, with the uplift at any section and at the base being redistributed once cracking has occurred.

16.9.3.3 *The Fenves-Chopra refined pseudo-static method*

The Fenves and Chopra (1987) method is a significant improvement on the Westergaard method. It uses data from more complex finite element analyses to allow the engineer to do a far more realistic analysis. In it:

- Dam response is included by taking from a suitable response spectrum the spectral accelerations at the first and selected higher modes for a nominated damping factor. For dams up to, say, 20 m in height, the dam's response may not be too critical.
- The water interaction model is more refined than the one often used (see next section).
- The flexibility of the foundations can be included.
- The effect of displacement of the crest upstream can be assessed.

The method gives an equivalent static loading applied to the upstream face of the dam that may be used to complete an analysis of the dam with the inclusion of the static loads (concrete weight, reservoir pressure, internal pore pressures and so on).

This method clearly has advantages over the Westergaard pseudo-static methods, but it does not allow for cracking of the base or at any other section that could change the dam's dynamic loading characteristics. For very stiff dams, cracking could see a substantial increase in the earthquake inertia loading, whereas in a large dam with a natural period of 0.3 seconds, the reverse might be the case.

Table 16.7 Hierarchy of Seismic Analysis Methods for Concrete Dams (ANCOLD, 2014).

i.	Simplified, linear elastic methods using a site specific response spectrum on a 2D model of the dam for pseudo static cantilever type stability analysis – generally only applicable to a concrete gravity dam;
ii.	Simplified, linear elastic methods using a site specific response spectrum on a 2D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam or for the upstream/downstream direction of a concrete buttress dam;
iii.	Linear elastic methods using a site specific response spectrum on a 2D or 3D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam (2D satisfactory for flat sloped abutments or buttress dams considering only upstream/downstream direction); 3D required for steep abutments), and for arch or buttress dams (considering full 3D action);
iv.	Simplified, linear elastic methods using a site specific time-history records of ground acceleration or velocity on a 2D or 3D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam (2D satisfactory for flat sloped abutments or buttress dams considering only upstream/downstream direction); 3D required for steep abutments), and for arch or buttress dams (considering full 3D action);
v.	Non-linear methods using site specific time-history records of ground acceleration or velocity on a 2D finite element model (FEM) of the dam and foundations so that cracking can be simulated – applicable to a concrete gravity dam or for the upstream/downstream direction of a concrete buttress dam; and
vi.	Non-linear methods using site specific time-history records of ground acceleration or velocity on a 3D finite element model (FEM) of the dam and foundations so that cracking can be simulated – applicable to a concrete gravity, an arch dam or for a concrete buttress dam;

One real advantage in this method is that if the stresses in the top of the dam are of concern, as was the case at Pine Flat and Koyna Dams, the stresses there can be quickly assessed.

FERC (2000) suggests that this method is a reasonable one to use and the authors would agree, subject to some careful review of the results.

16.9.3.4 *The US Corps of engineers method*

This method is presented in Guthrie (1986) and an excellent discussion on it is given in ANCOLD (1998).

This method goes somewhat further than the Fenves and Chopra (1987) method in that depending on the level of tensile stress developed, the engineer may have to undertake a limited FEM analysis. It would seem to be limited to some extent if one assumes, as the authors have suggested, that the dam foundation interface has no tensile strength. In that case, cracking will be a reality so unless the analysis can cope with a possibly varying natural period as the crack extends, then it would be better to go immediately to a full FEM analysis. If cracking at the dam foundation interface is limited so that the dam's natural period does not change very much, then the use of this method is reasonable.

16.9.3.5 *Finite Element Method (FEM)*

ANCOLD (2014) present a hierarchy of seismic analysis methods which is based on a similar table in ICOLD (2012). This is shown in Table 16.7.

Table 16.8 Analysis Method Appropriate for Consequence Category (ANCOLD, 2014).

	Consequence Category		
Analysis Methods	*Low*	*Significant*	*High and Extreme*
2D linear elastic, simplified response spectrum, cantilever analysis – gravity dam only – (i) in Table 16.7	Yes	Yes	Yes
2D linear elastic FEA, site specific response spectrum-gravity dam & buttress dam in u/s-d/s direction – (ii) in Table 16.7		Yes	Yes
2D or 3D FEA, site specific response spectrum- all concrete dams – (iii) in Table 16.7			Yes
2D or 3D linear elastic FEA, site specific accelerograms – all concrete dams – (iv) in Table 16.7)			Yes
2D non-linear FEA, site specific accelerograms – gravity dam & buttress dam in u/s-d/s direction – (v) in Table 16.7			Yes
3D non-linear FEA, site specific accelerograms – all concrete dams – (vi) in Table 16.7			Yes

ANCOLD (2014) suggest that the analysis method used for final analyses may be related to consequence category as shown in Table 16.8. ANCOLD (2014) recommend undertaking structural seismic analysis in a hierarchical manner. That is, analysis should start using simplified linear elastic methods and then progress as required, to more sophisticated methods. The authors agree with this approach.

As described in ANCOLD (2014) analysis will be either in the frequency domain where an earthquake response spectrum is used, or in the time domain where time-histories of velocity or acceleration are used.

For analyses in the frequency domain, obtain a response spectrum for the site (in the case of concrete dams, attenuation or amplification of accelerations through overburden is not likely to be an issue as it may be for embankment dams founded on soil) and for the relevant damping factor and design earthquake peak ground acceleration.

For analyses in the time domain, obtain a minimum of three sets (preferably five sets) of time-histories (two orthogonal horizontal time-histories and one vertical time-history in each set). It is likely that these time-histories will need to be synthesized from recorded time-histories by specialist seismologists. Use either amplitude scaling or spectral matching as appropriate, to ensure that response spectra produced from the synthesized time-histories match the design response spectrum.

If the aim of the analysis is to use three components of a recorded earthquake, then the amplitude scaling method should be used. In this method, the most likely of the three components that will be dominant is scaled to match the design response spectrum and the other two components are scaled by the same factor. Spectral matching does affect the phase of the time history so that the analyst would have problems about how to scale the other two components.

Either direct integration or modal superposition methods may be used to determine stresses in the dam. The former is the more accurate but requires considerably greater computer resources (computer memory and run time). The latter will be more

conservative as it uses a statistical approach (e.g. square root, sum of the squares) to combine the maximum stresses from the various modes. Consequently, it ignores the sign of the maximum stress for a particular mode.

The following are some comments on the application of FEM.

FEM has been used on many gravity dams since probably the late 1970s. The analyses have mostly been 2-dimensional analyses, although the authors know of several done in 3-dimensions. There have also been a number of arch dams analyses done, which have necessarily been 3-dimensional analyses. The most well known of these would be the USBR studies into the never-built large Auburn dam in California (USBR, 1977 and 1978). A recently completed analysis of an arch dam in South Australia, a dam with extensive gravity abutments, was able to allow for the effects of several non-linearities, including every vertical joint between blocks. The computation time was "heavy"; the result was an excellent study that pin-pointed the potential weaknesses in the dam to a large input earthquake load.

Well into the 1990s, most of the analyses by practising engineers used linear elastic FEM programs. Given the low level of stresses in a gravity dam, the assumption of linear elastic behaviour is reasonable, but cracking is hard to include in a full dynamic analysis. Now with computer programs available with more complex elements that will allow for crack joint opening, compressive loading, shear loading and sliding, the difficult question of cracking can be included in the FEM model.

Most FEM analyses are time-consuming. In the 1980s to early 1990s, it was lack of computer power; now (2014) it is very complex programs. In all cases, there is a cost associated with any FEM analyses. One almost flippant comment to the fourth author many years ago had and still has much truth in it; 'whatever you estimate, it will cost 3 times as much.'

Ideally, if, as the authors believe is the right approach, zero tensile strength is assumed at the dam/foundation interface, cracking must be modelled in this zone. Cracks can be modelled elsewhere, if the computer power is available. Once cracking is included, the engineer will be limited to time-history analyses in a step-by-step numerical integration process. At each small time interval, say 0.01 s, the dam is fully analysed under both the applied earthquake loading and the static loading in a number of iterations until equilibrium is satisfied. The analysis then moves to the next time step. With the analysis being an effective stress analysis, as are both the foregoing pseudo-static analyses, the pore pressures must be included. Permanent shear displacement can also be allowed for, but the difficulty here is that the analyst usually has to start with the residual strengths of the potentially sliding surfaces as input data. It is probably better to analysis the dam with peak strengths and to determine where displacement is likely to occur. That information would be the necessary information for the later post-earthquake analyses.

The mechanics of an FEM analysis are now reasonably well known and need not be repeated here. Nevertheless, there are some issues particularly related to dam analyses that need highlighting. They include:

(a) *Dam-reservoir interaction effects*

The USBR (1977, 1978) Auburn study in the late 1970s allowed for these effects to be included as 'added masses' fixed to the upstream face elements. The USBR used the Westergaard distribution.

The general opinion is that this approach is probably reasonable (Léger *et al.*, 1991), although, in the case of a 3-D analysis one would question the relevance of this added mass to a cross-valley earthquake. In the long run, an FEM analysis that includes the reservoir directly would be the ideal (see ANCOLD (1998), and Léger *et al.*, 1991 for more discussions).

Interestingly, in the recent FEM study on the arch dam in South Australia mentioned earlier in this section, the effort was made at the start to include the reservoir in the basic model. The results from this analysis were compared with the more standard approach of using the Westergaard added mass distribution. As the two sets of results were found to be quite close, the rest of the analyses were done using the Westergaard added mass approach, which reduced the computation time considerably.

(b) *Inclusion of the foundation rock*

Most analyses included the rock mass with fixities at least one 'dam height' away from the dam-foundation interface. For a static analysis, if one ignores the 'rebound effects' on excavation, the foundation has 'no mass'. The same approach is often taken in dynamic analyses, even though the resulting dam/foundation model is slightly stiffer than a model with rock with mass. For many dams the critical failure surfaces will be in the foundation so the foundation must be modelled. Some are now advocating foundation models that extend further from the dam, if the boundarary effects are to be avoided.

(c) *Inclusion of water pressure loads and internal pore pressures*

Most finite element analysis programs are not geared up to cope with internal pore pressures so generally engineers have opted to apply the reservoir water loads to the dam's upstream face and any exposed dam faces of cracks. The final effective stresses within the body of the dam are found by adding the pore pressures to the calculated stresses from the FEM analysis (compressive stresses usually taken as negative). Some engineers apply the water load to the dam's face and uplift as an external load to the base.

This approach does not allow the correct results to be achieved in a step-by-step numerical integration procedure. To overcome this shortcoming, analyses can be done to define the seepage pressure forces. Typically, for an FEM package that has a heat flow model contained within it, this heat flow program, with assumed boundary conditions (for example, the very conservative assumption of full head at upstream face and zero at downstream face), can be used to calculate these seepage forces. The latest software, such as DIANA, can do this seepage analysis more directly. In the end, the total water pressure loads are included as body forces in much the same way as gravity forces.

If the stress distribution is required within the foundations, then similar body force distribution can be found. The pre-existing groundwater conditions need to be included – for example the original ground water surface could be assumed as coincident with the ground surface (probably appropriate to a dam section near or at maximum section).

The pore pressures are usually assumed not to change during the earthquake. Some have argued that this assumption should not apply to a crack that opens and closes cyclically, but most design guidelines recommend that the 'unchanged' pore pressure model is acceptable.

(d) *Shear strength conditions at crack elements*

Ideally, the chosen shear strength law should have peak strength used until shear displacement starts at which point the strength would drop to the appropriate residual strength law. The residual strength would apply thereafter. One would have to nominate a complete shear failure path, usually a planar surface on the section or at the dam-foundation interface. Movement would not be assumed until all crack elements on that surface have reached the peak strength.

This relatively complex model, as far as the authors know, may not be readily available in FEM packages. Accordingly, the procedure is usually to adopt the residual strength as the shear strength law from the start of the analysis. The calculated permanent shear displacement for most analyses would therefore tend to over-estimate the shear displacement on any selected surface. Alternatively the peak strengths can be used and sliding surfaces then identified from the results for later input into the post-earthquake analyses.

(e) *Damping*

Damping, usually as a percentage of critical damping is an important input for any FEM analysis. Some practising engineers choose the damping factor at the low value, say, 5% of critical damping, arguing that the dam's stress levels are relatively low so damping will be corresponding low. Others argue that any crack development, permanent shear displacement, the presence of defined contraction joints between blocks and similar features warrant selection of a damping factor approaching 10–12%, particularly for MDEs with a PGA above 0.3 g. The authors favour the selection of damping factors close to 10% for an MDE analysis of gravity dams, when the peak ground accelerations of the selected design event, not necessarily the scaled time-histories, are higher than about 0.35 g (it would be reasonable to use lower damping factors for arch dams). Strictly, damping should be varied throughout the analysis, but the simple approach of a constant factor is generally applied from the start.

16.9.3.6 *Design earthquake input motion*

Regardless of whether the response spectrum or the accelerogram (acceleration time-history) input is the basis for the analyses, several selected sets of input data should be used. Advice should be sought from a specialist seismologist on this input data.

16.9.3.7 *Should vertical ground motion be included?*

There is no reason that vertical ground motion should not be included in any analysis. Given that the response effects to vertical ground motion are likely to be very small, the simple approach of reducing 'gravity' by the peak vertical acceleration would be appropriate for the pseudo-static methods to account for this motion. For the more complex FEM analyses, the engineer should apply to the model a measured vertical accelerogram along with its companion horizontal accelerograms. The vertical motion does not get applied to an 'added' mass that is simulating the dam-reservoir interaction.

The advice of an experienced seismologist should be sort as to how best to handle vertical ground motion and to what level that motion should be.

16.9.3.8 *Reservoir level variation*

If a reservoir only gets to full supply level infrequently, a lesser reservoir level may be justified for analysis.

16.9.3.9 *What do the results of analyses mean?*

The engineer doing analysis of a concrete dam must sooner or later face up to this question. "*Should I check the stability of the dam at its peak displacement?*" "*What does a permanent shear displacement of 100–200 mm mean?*"

In the case of the pseudo-static methods, if reasonably high pgas are used, the dam will inevitably be shown to be unstable. The method, by its very nature, demands that the dam's stability be checked.

FERC (2000) states that:

> *"In a departure from the way the FERC has previously considered seismic loading, there is no longer any acceptance criteria for stability under earthquake loading. Factors of safety under earthquake loading will no longer be evaluated. Acceptance criteria are based on the dam's stability under post earthquake static loading considering damage likely to result from the earthquake. The purpose of considering dynamic loading is to determine the damage that will be caused so that this damage can be accounted for in the subsequent post earthquake static analysis."*

and later:

> *"Because of the oscillatory nature of earthquakes, and the subsequent structural responses, conventional moment equilibrium and sliding stability criteria are not valid when dynamic and pseudo dynamic methods are used. The purpose of these investigations is not to determine dam stability in a conventional sense, but rather to determine what damage will be caused during the earthquake, and then to determine if the dam can continue to resist the applied static loads in a damaged condition with possible loading changes due to increased uplift or silt liquefaction."*

FERC (2002) approach is that the analyses are to show if and where there could be problems, such as permanent displacement, and how these problems might affect the dam, for example, extensively cracking the dam, seriously damaging prestressed anchors, completely cutting foundation drains and so on. From these results, one would decide what weaknesses have been created and whether or not this weakness could influence the dam's post-earthquake condition. Partial or full loss of prestressed anchors, change in drainage conditions, extensive cracking and loss of shear strength would be typical of the impacts in a post-earthquake analysis.

The FERC recommendations have much merit. However it should be recognised that a simple structure subjected to ground shaking could collapse during the earthquake, not just afterwards. Perhaps the key point is that most gravity dams are not simple structures; for example, there could be 3-D action or significant differences, even reversals, in displacement along the axis at any particular time interval. The authors' preference for new dams would be to have a theoretically stable dam at all stages of an analysis, including, in the case of a step-by-step numerical integration FEM, at the time-step for the maximum displacement of the dam.

There has been a considerable advance in the analytical tools over the last 10 years, with, no doubt, a corresponding increase in the time and costs of the analyses, but a complete package that allows for the progressive loss of shear strength along a possible defect, for example, still does not seem to be available. Regardless of the complexity of the dynamic analyses done, post-earthquake analyses must be done to complete the assessment of the dam under earthquake loading.

Calculated permanent displacements of, say, 100 mm and above seem at odds with these large, stiff structures whose maximum displacement under elastic loading are probably only up to 10–15 mm. Such large permanent displacements imply the development of considerable distress in the dam and much remedial work if it is to be retained.

Engineers should not be too carried away with any of these analyses, no matter how complex. The words of Newmark and Rosenbleuth (1971) from some 40 years ago still hold true:

> *"We must also face uncertainty on a large scale, for it is our task to design engineering systems – about whose pertinent properties we know little – to resist future earthquakes and tidal waves – about whose characteristics we know even less."*

16.9.3.10 *Post-earthquake analyses*

As emphasised by FERC (2002) these analyses are vital to any gravity dam that could be subjected to earthquake loading. They should be done carefully, with conservative strength factors allowing for the deformations and cracking induced by the earthquake and proper consideration of changed uplift pressures.

16.9.3.11 *Dams on rock foundations with potentially deep-seated failure mechanisms*

For most of the foregoing discussions, the tacit assumption has been that the dam-foundation interface zone is the critical foundation component in the analyses. If there are known defects and weaknesses, such as existing bedding surface shears, that could give rise to a potential deep-seated failure, the analytical problem increases again in complexity.

Without going into highly complex coupled foundation-dam analytical models, the likely steps are:

- The dam-foundation mechanism presumably can be proven to safe under static loads. Therefore calculate the dam's maximum response base moment and shear using the methods just described, and apply them to the potential foundation failure mechanism. If there is an adequate safety margin, with conservatively chosen strength parameters, the dam-foundation interface can be accepted as the critical foundation zone.
- If movement does occur on the deep-seated failure mechanism, a yield acceleration should be calculated, allowing for the dam's loading, and a Newmark analysis done to estimate the total permanent displacement. This type of approach may be reasonable as during sliding on the foundation mechanism, the impact of the earthquake will not be transmitted through to the dam above (USBR 1989).



Figure 16.30 Damage to Shikang dam in Taiwan in 1999. The vertical displacement was about 9 m and the horizontal displacement was about 1.5 m into the abutment.

The difficulty posed by this particular problem clearly illustrates the need for very careful geological exploration and assessment of the damsite. Any very stiff structure like a gravity dam could be under serious threat if it were to be subjected to potential displacements from ground movement.

16.9.3.12 *Dams on foundations that could be subjected to ground displacement*

None of the above methods can hope to cope with major ground dislocation. Clyde dam in New Zealand does include a design feature to allow for some differential displacement, but even that feature would not handle the displacement of 9 m vertically and 1.5 m laterally that occurred at Shikang dam in Taiwan in 1999 (see Figure 16.30).

16.9.4 Concluding remarks

The analysis of a gravity dam for earthquake loading will inevitably present problems to the people involved. This situation seems even more of an issue today as engineers adopt high pgas and use analyses that really do not simulate what is going to happen. Perhaps the best thing to do is to make sure the dam is conservatively designed for its static loading, for example, use shear strength parameters for rock and concrete and concrete tensile strengths that are not excessively high and provide proper drainage, and to take simple defensive precautions, for example, those listed in ANCOLD (1998). For the comfort of all concerned, the record of gravity dams under earthquake loading is good (Hinks and Gosschalk, 1989). But perhaps it is because few have been subject to very large earthquakes.

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Chapter 17

Foundation preparation and cleanup for embankment and concrete dams

17.1 GENERAL REQUIREMENTS

17.1.1 Embankment dams

The degree of foundation preparation which is necessary for a dam embankment depends on the:

- Type of dam.
- Height of dam and the consequences of failure.
- Topography of the dam site.
- Erodibility, strength, permeability, compressibility of the soil or rock in the dam foundation.
- Groundwater inflows to excavations.
- Climate and river flows during construction.

Foundation preparation for the *'general foundation'*, i.e. the foundation beneath the bulk of the embankment, is quite different from that for the *'cutoff foundation'*, i.e. the foundation under the earthfill core of an earth and rockfill dam prepared to cutoff standard, or the plinth of a concrete face rockfill dam.

Figure 17.1 shows the terms used in this chapter.

The objectives are:

- *General foundation*: To remove low strength and compressible material to provide a foundation of adequate strength and compressibility to support the embankment. In most cases, but not always, the general foundation will be of higher strength than the embankment and will not dictate the embankment stability. In most cases permeability will not be a critical factor. Liquefiable materials may need to be removed or treated.
- *Cutoff foundation*: To remove highly permeable and erodible material below the general foundation level to provide a low permeability non erodible foundation (consistent with the design of filters and drains on the foundation). In many cases, e.g. in soil foundations or rock foundations which are permeable to great depth, a low permeability non-erodible cutoff foundation cannot be economically achieved by excavation alone and other design measures are required. These are discussed in Chapter 10.

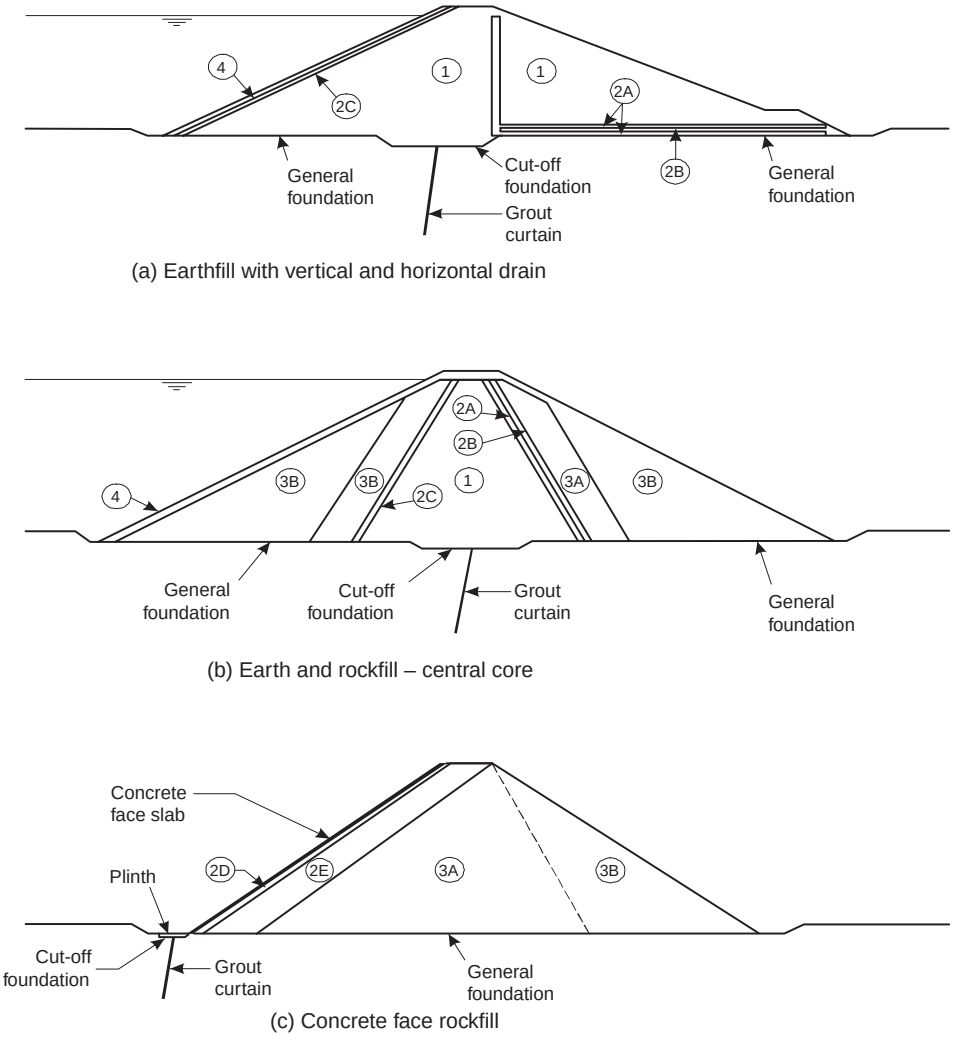


Figure 17.1 Terms used for foundation preparation for embankment dams: (a) Earthfill with vertical and horizontal drains, (b) Earth and rockfill, (c) Concrete face rockfill.

17.1.2 Concrete dams

The degree of foundation preparation for a concrete dam depends on the:

- Type of dam (gravity, arch, buttress)
- Height of the dam and the consequences of failure
- Topography of the dam site
- Loads imposed on the foundation, and the strength and compressibility of the soil or rock in dam foundations, allowing for the detailed geological structure, including bedding, jointing, and the presence of shears and faults.

– Erodibility and permeability of the soil or rock in the dam foundation.
– Groundwater inflows to excavations.
– Climate and river flows during construction.

Figure 17.2 shows three arrangements for foundation preparation under concrete gravity dams. (a) shows the conventional arrangement, where the foundation is all taken to a uniform requirement, (b) shows the case where a low strength defect exists in the rock (e. g. low strength bedding surface) and the foundation is taken below the defect and (c) shows a foundation where 'keys' have been used to take the critical failure surface deeper into the foundation. 'Keys' or cutoffs may be required to prevent undercutting of the downstream toe by the spillway flow.

In each case A-B is prepared to concrete dam foundation standard (A-B′ if the additional concrete in Figure 17.2b is required). A lesser standard may apply under the stilling basins, although such would depend very much on the pounding from the rapidly varying hydrodynamic pressures within the basin that might be expected during the spillway discharges, or between the keys in Figure 17.2c.

Grout holes and drainage holes are usually provided as shown in Figures 17.2a and 17.2b to control uplift pressures beneath the dam. For smaller dams contact drains, as shown in Figure 17.2c have been used. These drains are ineffective except at possibly the foundation contact as discussed in Chapter 16.

In general terms, after normal treatment by grouting and drainage, the rock foundations must:

– Be of adequate strength, stiffness modulus and durability to support the dam loads, without excessive deflections or settlements, under reservoir empty, full and all operating conditions.
– Have sufficient shear strength to provide an adequate factor of safety against sliding downstream under all operating conditions.
– Be sufficiently impermeable and non-erodible to prevent excessive leakage beneath the dam.
– For dams with overfall spillways, or subject to overtopping during extreme flood events, be able to withstand the impact of flood overflows at and near the toe, without erosion which could endanger the dam. It is now recognised that stilling basins under flood flows are likely to be subjected to severe hydrodynamic load variations at frequencies not unlike those measured in ground shaking during a large earthquake.

For concrete gravity dams founded on soil (as many smaller dams and 'weirs' are), the requirements are essentially the same as for rock, but the foundations are weaker and more compressible than rock, and are erodible, so the design must account for this.

17.1.3 Definition of foundation requirements in geotechnical terms

In all cases it is important to define the requirements in geotechnical terms which can be identified in the field during construction. This is discussed in Sections 17.5 and 17.8.

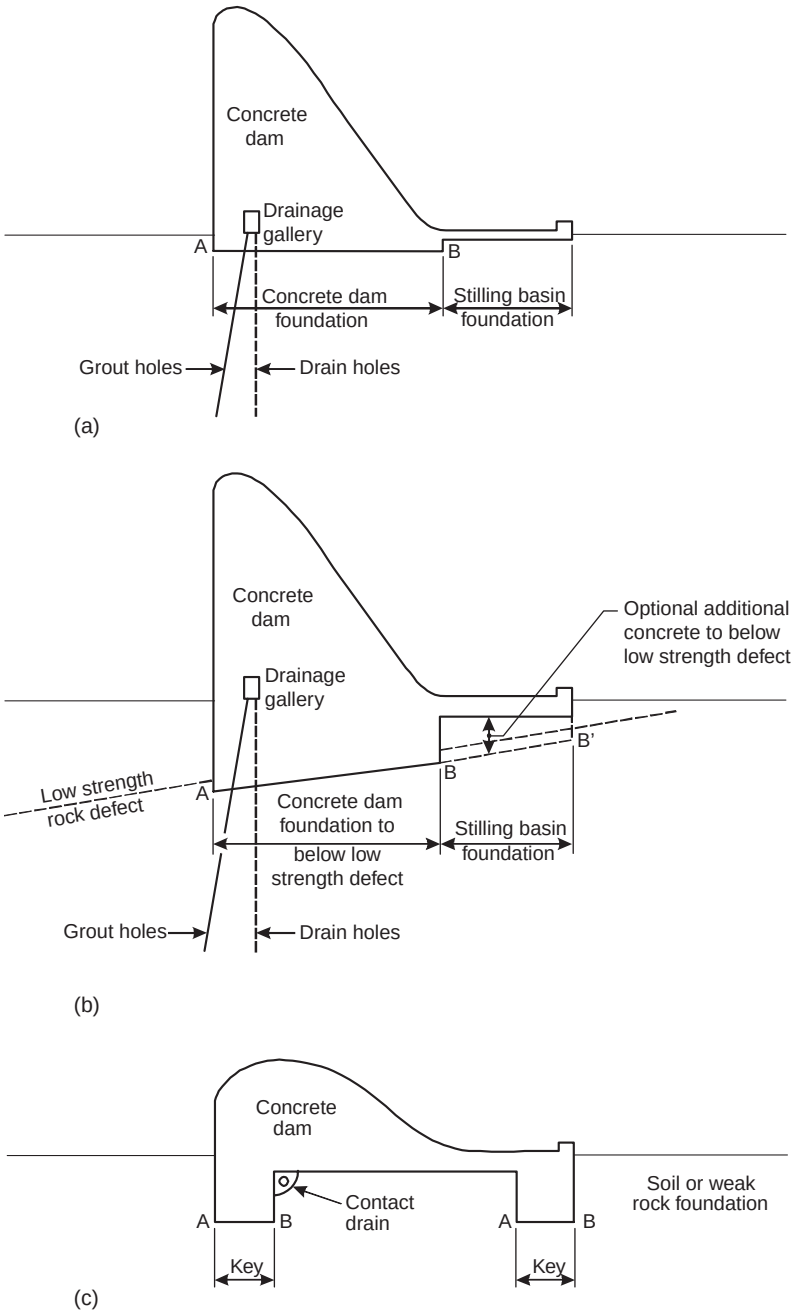


Figure 17.2 Foundation preparations for concrete gravity dams: (a) Conventional arrangement for dam on rock foundation, (b) Dam on rock foundation with low strength defect in foundation, (c) Dam with 'keys'.

It is likely that in most cases excavation will proceed in stages, with sufficient clean up to identify whether the geotechnical requirements have been achieved.

17.2 GENERAL FOUNDATION PREPARATION FOR EMBANKMENT DAMS

The requirements for preparation of general foundations for embankment dams are described below.

17.2.1 General foundation under earthfill

17.2.1.1 *Rock foundation*

- Remove topsoil and weak compressible soil. In most cases this will involve removal of colluvial soil and rock, including boulders, to expose an *in-situ* rock foundation which may be extremely weathered, variably weathered or in some cases fresh (unweathered). An adequate foundation can usually be identified by near blade refusal of a small bulldozer or excavator.
- Where there are unfavourably oriented weak seams in the rock, e.g. bedding surface shears or landslide rupture (slide) surfaces, these will influence stability and may need to be removed, or the design modified to accommodate them.
- Slope modification may be necessary as described in Section 17.6.

The surface should be cleaned of loose soil and rock before placing earthfill, e.g. with a grader, backhoe or excavator. Intensive cleanup is not generally required. It may be desirable to moisten the surface prior to placing earthfill to maintain adequate moisture in the earthfill for compaction, and treat open joints to prevent erosion of the soil into the joint. It should be noted that more clean-up may be needed in localised strips or the like to allow the finished foundations to be mapped.

17.2.1.2 *Soil foundation*

- Remove topsoil and weak compressible soil consistent with the assumptions made for stability and settlement analysis and design.
- Where soils are fissured, or have landslide rupture (slide) surfaces present, these will influence stability and may need to be removed, or the design modified to accommodate them.
- Slope modification may be necessary as described in Section 17.6.
- The surface may be proof rolled with a steel drum or tamping foot roller to assist in locating weak and compressible soil. Rolling to a specified compaction requirement should not be necessary.

The surface should be cleaned of loose soil and rock prior to placing earthfill, e.g. with a grader, backhoe or excavator. Intensive cleanup is not generally required. It may be desirable to scarify and moisten the surface prior to placing the first layer of soil to assist in 'bonding' the embankment to the foundation.

17.2.2 General foundation under rockfill

Rockfill will generally be underlain by a rock foundation (or by a horizontal filter drain, in which case refer to Section 17.2.3):

- Remove topsoil, soil and weathered rock which has strength lower than the rockfill. In most cases this will involve exposing distinctly weathered rock which is very weak or weak. An adequate foundation may be identified by blade refusal of a bulldozer but for high dams light ripping may be necessary.
- Where there are unfavourably oriented weak seams in the rock, e.g. bedding surface shears or landslide rupture (slide/surfaces), these will influence stability and may need to be removed, or the design modified to accommodate them.
- It is unlikely that slope modification will be required under rockfill other than overhanging cliffs being treated by removing the overhanging rock, or filling the re-entrant below the overhang with concrete.
- The surface should be cleaned of loose soil and rock with a bulldozer, grader, backhoe or excavator sufficient to ensure the rockfill is supported on the rock foundation. Intensive cleanup is not required and if there is loosened rock of good quality this may be left in place, localised clean-up should be done along selected strips so that the finished foundation can be properly mapped.

If there are wide, erodible seams in the rock, oriented such that they are likely to carry seepage water from the storage, it may be necessary to cover them with concrete or shotcrete under the upstream rockfill, and by a filter layer under the downstream rockfill prevent erosion.

17.2.3 General foundation under horizontal filter drains

Horizontal filter drains will generally only be required if the foundation is soil or erodible rock:

- Remove topsoil and weak compressible soil consistent with the assumptions made for stability, settlement and particle size used for design of the filter.
- Where soil or rock in the foundation is fissured or has landslide rupture (slide) surfaces or unfavourably oriented bedding surface shears are present, these will influence stability and may need to be removed, or the design modified to accommodate them.
- Slope modification should not be required except to remove overhangs. However, if earthfill is to be placed on top of the filter drain, slope modifications may be needed as described in Section 17.6.
- The surface should not be rolled prior to placing the filter. Rolling will destroy the soil structure and reduce the permeability, making it more difficult for seepage water to flow into the filter drains. It is desirable for the foundation seepage to flow into the filter drain so erosion is controlled, rather than being forced to emerge downstream of the toe of the embankment in an uncontrolled manner. This is an important issue, often not well appreciated by construction personnel and even some dam engineers.

- On low strength rock and on soil, trafficking with earthmoving equipment will continuously break up the surface, necessitating final cleanup with an excavator or backhoe working away from the cleaned up area. This is particularly a problem with weathered schists, phyllites and similarly fissile rocks which break down very easily under equipment. Once cleaned, the filter should be dumped on the cleaned up surface and spread onto the surface without equipment trafficking directly onto the foundation
- Immediately before placing the filter material, the surface should be cleaned of loose dry and wet soil and rock. This may necessitate intensive work using light equipment and hand methods. Final cleanup should involve an air or air water jet to 'blow' away loose material. In many cases an air-water jet will be too severe and cause erosion.

17.3 CUTOFF FOUNDATION FOR EMBANKMENT DAMS

17.3.1 The overall objectives

The requirements for excavation below general foundation level to achieve a suitable cutoff foundation are described below. It should be noted that each case will be determined on its merits and often there will be trade-offs between the following:

- The desire to achieve a low permeability foundation.
- The depth and hence volume and cost of excavation required to achieve a cutoff.
- The extent of grouting planned in the foundation below the cutoff.
- The protection downstream to control foundation erosion, e.g. filters over the surface of the foundation.

High groundwater levels, e.g. in alluvial soils, may determine the practical depth to which a cutoff may be taken, since dewatering is usually expensive. The guidelines given below, therefore, are stating the 'desirable' requirements rather than what may be practicable in some cases.

It should also be noted that many of the features described below cannot be readily identified in the base of the cutoff excavation, but are often more apparent in the sides of the excavation. Hence it is normal to require progressive excavation and cleanup, with a minimum excavation depth of about 0.5 m below general excavation level to confirm that the requirements have been achieved.

Having defined the cutoff foundation, the further objective when placing the earthfill, is to ensure a low permeability contact between the earthfill and the foundation. This is done by:

- Modifying the shape of the foundation surface to provide a surface suitable for earthfill compaction (Section 17.6).
- Using compaction equipment which facilitates compaction but does not damage the foundation (Section 14.2).
- Using more deformable, less erodible soils adjacent to the contact – i.e. using higher plasticity soils placed wet of optimum water content (Section 14.2).
- Filling erodible seams or open joints in the rock with concrete prior to placing the core material (Section 17.6).

17.3.2 Cutoff in rock

- Remove rock with open joints and other fractures which would otherwise lead to a highly permeable structure. In many cases this will result in a foundation which has permeability generally less than say 15 Lugeons. For large, high hazard dams it would be normal to aim for a foundation with permeability generally less than 7 Lugeons. However it is emphasized that this may not be practicable in many situations and higher permeability rock will be left in the foundations.
- Remove rock with clay infilled joints, roots etc., which may erode under seepage flows to yield a high permeability rock. This is particularly important where the clay has been transported into the joints and/or is dispersive, as this indicates its likely erodibility.
- Carry out slope modification and treatment as described in Section 17.6.
- Where the exposed rock is susceptible to slaking by wetting and drying (e.g. many shales) or breakdown under trafficking, it should be covered with a cement-sand grout (thickness usually <10 mm to 25 mm), pneumatically applied mortar (shotcrete) or concrete (minimum thickness 50 mm and preferably not less than 150 mm). Generally this should be done immediately the foundation is exposed, but may be done after a second cleanup immediately before placing the earthfill core. If cement-sand grout is used, it may crack under the trafficking of equipment placing the earthfill and it may have to be removed immediately prior to placing the earthfill.
- Remove from the surface all loose soil and rock, and debris from grouting (using light equipment and with an air or air-water jet). Hand cleanup may be necessary. The surface may need to be moistened immediately before placing earthfill to maintain the earthfill moisture content.
- If the rock in the floor or the sides of the cutoff trench displays open joints or other features which would allow erosion of the earthfill into them, it should be cleaned of loose material and covered by a cement-sand grout, pneumatically applied mortar or concrete. This is particularly critical on the downstream side of the cutoff trench. If there are only a few such features they might be treated dentally. The foundation contact surface for the narrow earthfill core for the 160 m high Thomson Dam was fully shotcreted or covered by backfill/slope treatment concrete.

Under no circumstances should the surface of the cutoff foundation be rolled, even if it is a weathered rock. Rolling will only disturb the rock leading to a higher permeability and potentially erodible material.

Figures 17.3 to 17.8 show cutoff foundation cleanup, treatment of seams and open joints for Dartmouth, Kenyir and Blowering Dams.

17.3.3 Cutoff in soil

- Remove soil with open fissures, open joints, roots, root-holes, permeable layers (e.g. sand and gravel) and other permeable structure (e.g. leached zones in lateritic soils).



Figure 17.3 Abutment of Dartmouth Dam showing cleanup in cutoff foundation, and construction of the grout cap (courtesy of Goulburn-Murray Water).

- Remove dispersive soils if possible.
- Carry out slope modification as described in Section 17.6.
- If the soil on the sides of the cutoff trench displays permeable layers or features, which would allow erosion of the earthfill into them, it should be trimmed and cleaned and covered with a filter layer or layers which are designed to control such erosion. Details of such treatment are shown in Figure 10.8. In extreme cases it may be necessary to also cover the slope with pneumatically applied mortar.
- Remove loose and dry soil and other debris with light equipment, possibly with the aid of an air jet. The base of the cutoff should be watered to within 2% dry and 1% wet of optimum water content and rolled before placing the first layer of fill, to compact any soil loosened by the construction work. If necessary it should be disked or tyned with a grader to facilitate the moisture conditioning.



Figure 17.4 Dental concrete in the cutoff at Kenyir Dam (courtesy of Snowy Mountains Engineering Corporation).

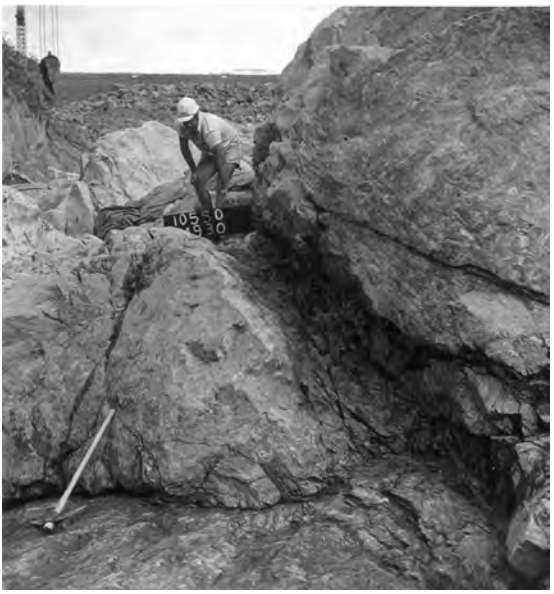


Figure 17.5 Excavation of sheared zones prior to filling with dental concrete in the cutoff foundation of Dartmouth Dam (courtesy of Goulburn-Murray Water).



Figure 17.6 Cutoff foundation for Blowering Dam showing open joints being treated with pneumatically applied mortar, but no treatment for other joints (courtesy of Snowy Hydro).



Figure 17.7 Application of pneumatically applied mortar on seams in the cutoff foundation of Blowering Dam (courtesy of Snowy Hydro).



Figure 17.8 Left abutment of Blowering Dam after treatment of large fault zone and weathered seam (courtesy of Snowy Hydro).

17.4 WIDTH AND BATTER SLOPES FOR CUTOFF IN EMBANKMENT DAMS

The width of cutoff trench adopted depends on:

- the rock or soil quality at cutoff foundation level.
- the height and consequences of failure of the dam.
- the type of embankment.
- the extent to which the cutoff surface is treated to control potential erosion of the core into open defects in the foundation.

It is not always necessary to treat the whole of the contact between the earthfill core and the foundation as 'cutoff', and only part of that area is excavated to cutoff foundation requirements. Figure 17.9 shows a central core earthfill embankment and for this and other earth, and earth and rockfill embankments the following guidelines are given.

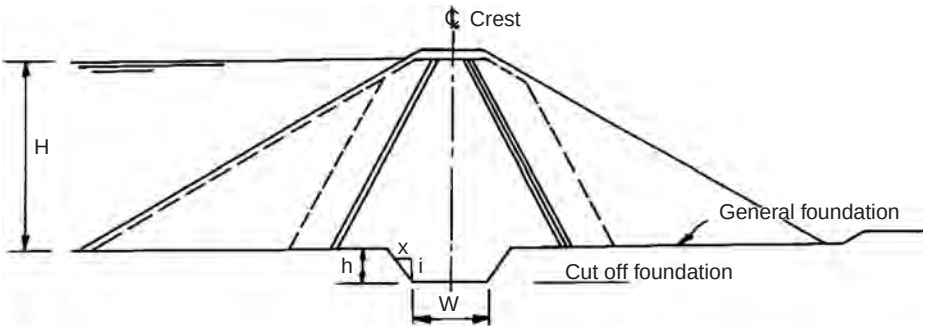


Figure 17.9 Cutoff trench details.

17.4.1 Cutoff width W

For a cutoff in rock, the width depends on the quality of the rock, e.g. for low permeability non erodible rock, W/H may be as low as 0.25 but would usually be 0.5 or more. There have been cases where W/H = 0.1 has been used. For concrete face rockfill dams W/H ≈ 0.05 to 0.1 is quite common (ICOLD, 2010). For cutoff foundations on more erodible rock with open defects W/H may be taken as the full core width, commonly >0.5H and up to 1H.

For large high consequence of failure dams it is common to be conservative and the full core contact is taken to cutoff foundation conditions regardless of the rock foundation.

The cutoff width should be not less than 3 m even for small dams (the width of excavation and compaction equipment) and preferably 6 m.

For a cutoff in soil, it would be common to make W/H ≈ 1. The idea is to give a lower seepage gradient for these more erodible conditions.

It is emphasised that, if the depth to an acceptable cutoff is excessive, these guidelines may be waived, provided other measures are adopted, e.g. multi-line curtain and consolidation grouting, cutoff walls constructed under bentonite and erosion protection in the base and sides of the cutoff.

Regardless of the width of the cutoff open defects in rock, or coarse soils into which the core may erode must be treated as described in Section 17.6.3 to prevent erosion.

17.4.2 Batter slope

The batter slopes for the cutoff trench should be:

- Not steeper than 0.5H:1V, and preferably 1H:1V so that the earthfill can be compacted against the sides of the trench.
- Sufficiently flat to avoid arching effects in the cutoff trench. As a guide, if h/W > 0.5, the batter slopes should be reduced to 1H:1V or 1.5H:1V.
- The batter slopes may be determined by stability during construction, e.g. by joints or other defects in rock, the strength of the soil or the presence of groundwater.

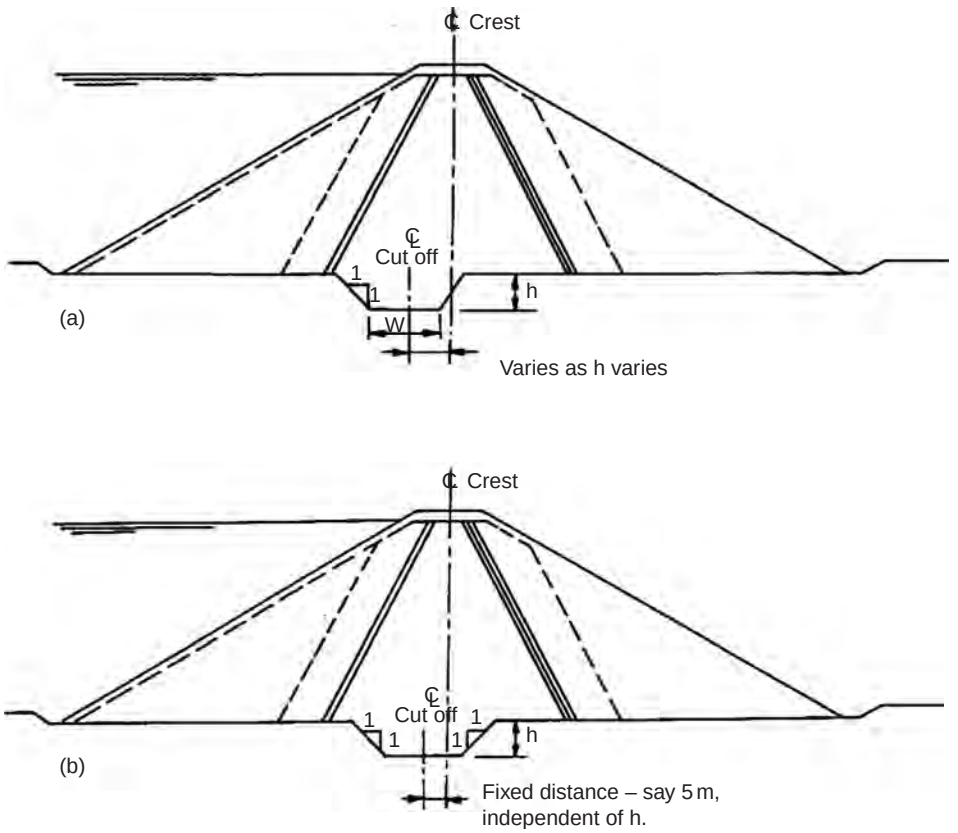


Figure 17.10 Cutoff trench layout: (a) Incorrect set out, (b) Correct set out.

For soil, it would not be uncommon to require batter slopes between 1H:1V and 1.5H:1V and possibly less steep.

17.4.3 Setting out

For setting out and construction purposes it is desirable to position the cutoff trench a fixed distance from the dam centreline, or by means of a series of fixed points. Figure 17.10a shows the effect of positioning the trench at the upstream contact of Zone 1 and the foundation. The result is a 'footprint' of the cutoff trench which is curved and more difficult to construct than Figure 17.10b which is positioned relative to the centreline.

The actual width of the base of the trench is best set as a fixed width or widths and made narrower up the abutments where the effective height is less.

It should be remembered that the actual depth of the cutoff trench will be determined by geotechnical factors and is not known accurately before the start of construction. It is necessary to set out the batter pegs for the excavation based on an

assumed depth of excavation. It is wise to be somewhat conservative in doing this, particularly if a minimum width (3 m) trench is being used, so that it is not necessary to restart excavation from the surface in the event that the actual depth to cutoff foundation is greater than expected.

17.5 SELECTION OF CUTOFF FOUNDATION CRITERIA FOR EMBANKMENT DAMS

The selection of cutoff foundation criteria for a project will depend on the particular site and embankment under construction. The following useful hints assist in this process:

- Never specify a depth of cutoff trench, i.e. depth below general foundation. The requirements for cutoff are geological and geotechnical and the depth to achieve these will vary. It is, however, appropriate for the geotechnical engineer/engineering geologist, in consultation with the embankment designer, to estimate the likely depths as a guide for the construction personnel.
- Do not specify a weathering grade alone as a requirement to achieve a suitable foundation. The requirements for cutoff foundations are essentially those of permeability (which for rock is determined by the presence of open joints and other defects) and erodibility, and a weathering grade alone (e.g. slightly weathered rock) may not be adequate or may be too conservative. If weathering grade is used as one of the requirements, it should be clearly defined in the documents what the different weathering grades are.
- Planning and selection of cutoff foundation requirements should use all available information. This information includes borehole log data - rock type, degree of weathering and fracture log, the Lugeon water pressure test results, presence of limonite and clay infill joints, seismic refraction data; and bulldozer trench/test pit information.

It will often be found that there is some correlation between these data which will allow logical decision making, e.g.:

- In developing general excavation requirements the soil/extremely weathered rock will have low seismic velocity (300–1000 m/sec) and will commonly coincide with blade refusal in the dozer trench. Hence the low seismic layer boundary may form a useful means of extrapolating between 'hard data' in the trenches and boreholes.
- The base of highly fractured rock, high-moderate Lugeon value, and medium seismic velocity (e.g. 1400–2000 m/sec) will sometimes coincide and indicate a reasonable cutoff level. Again the seismic data allows interpolation around the site.
- The base of limonite staining in joints often coincides with the base of high Lugeon rock. If weathering grade is related to this staining there may also be a correlation between weathering grade and Lugeon value.

Selection of planned cutoff criteria is sometimes done by laying out the drill core and the design engineer and engineering geologist observing the core, logs and other data and selecting the cutoff level or levels and the associated criteria there and then.

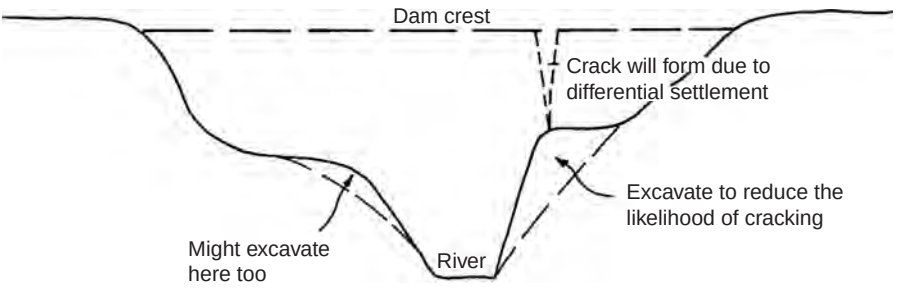

Dam crest
Crack will form due to differential settlement
Excavate to reduce the likelihood of cracking
Might excavate here too
River

Figure 17.11 Slope modification in the cutoff foundation to reduce differential settlement and cracking of the earthfill core.

Often there will be more than one choice, with a 'better' cutoff being available at depth and an 'adequate' cutoff closer to the surface. The 'adequate' cutoff may require a wider cutoff trench, more grouting, a grout cap, downstream filter protection etc., the need for which has to be offset against the shallower depth.

Similarly, for dams on a soil foundation, the cutoff depth and criteria can often be best established during the logging of test pits.

If this approach is used, the geological features which were used to select the cutoff can be described and readily identified in the construction process. The engineering geologist and design engineer, who make the design decision, should be present during construction to define and confirm those conditions in the cutoff trench.

17.6 SLOPE MODIFICATION AND SEAM TREATMENT FOR EMBANKMENT DAMS

17.6.1 Slope modification

Excavation of near vertical or overhanging surfaces and/or backfilling with concrete is required.

For general foundation:

– To allow compaction of earthfill and avoid cavities under rockfill due to overhangs in rock in the abutment.

For cutoff foundation:

– To allow compaction of earthfill to give a low permeability contact between the earthfill and the foundation.
– To maintain positive pressure of the earthfill on the abutment.
– To limit cracking of the earth core due to differential settlement over large discontinuities in the abutments, e.g., as shown in Figure 17.11.

To allow earthfill to be compacted and to maintain positive pressure on the abutments, steep small scale foundation surfaces should be flattened to about 0.5H to

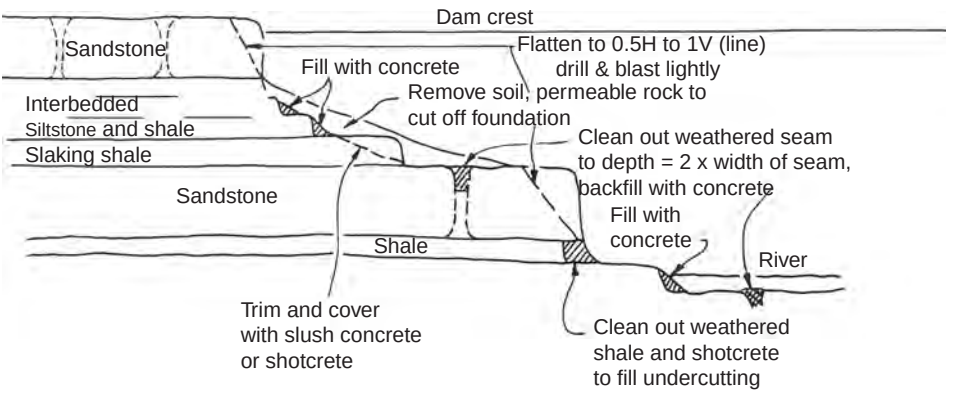


Figure 17.12 Slope modification and seam treatment in the cutoff foundation for interbedded sandstone and siltstone, near horizontal bedding.

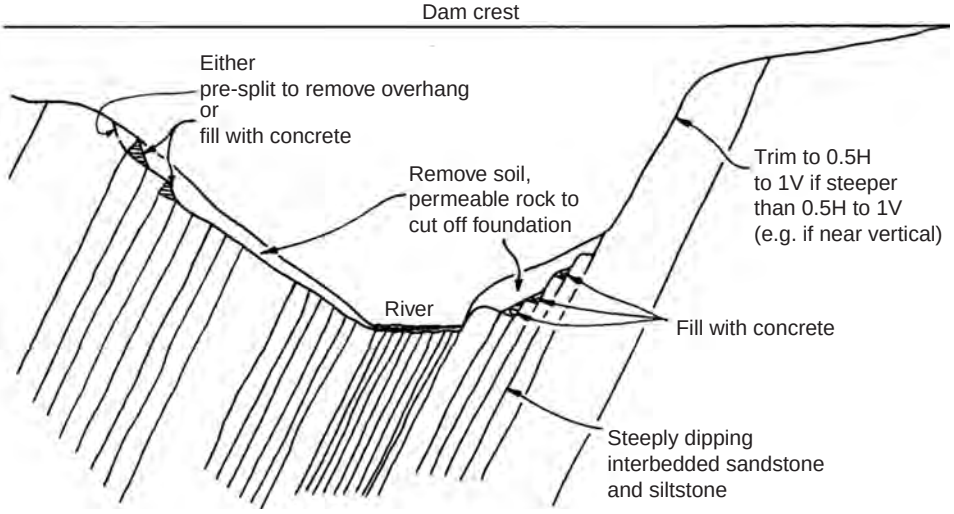


Figure 17.13 Slope modification in the cutoff foundation for interbedded sandstone and siltstone, steeply dipping bedding.

1V either by excavation or by backfilling with concrete. Figures 17.12 and 17.13 give examples of this type of treatment. Some authorities require a flatter slope, e.g. Thomas (1976) and Wallace and Hilton (1972) suggest the use of 0.75H:1V. Others, e.g. Acker and Jones (1972), accept steeper slopes (0.25H:1V). USBR (1984) suggest 0.5H:1V. The policy for Thomson dam was to be guided by the persistence of any slopes, particularly with slopes within F 30° to the upstream-downstream direction for which 1H:1V slope was sought. Significantly high slopes were made not steeper than 0.75H:1V. Sharp changes in grade near the tops of any slope were 'rounded out'. Thomson is a very large dam and it might be argued greater conservatism is warranted

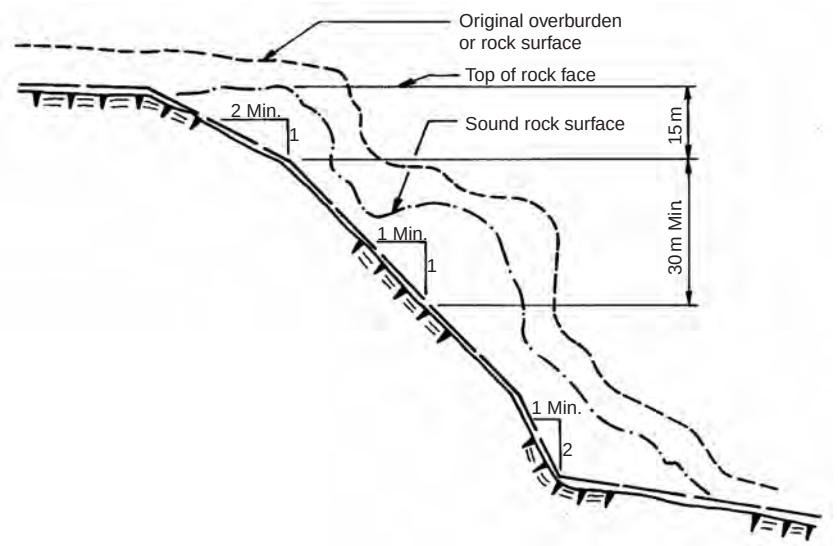


Figure 17.14 Bennett Dam, core abutment excavation requirements (Pratt *et al.*, 1972).

in these cases. The authors are of the opinion that 0.5H:1V is satisfactory provided the earthfill is compacted wet of optimum with rubber tyred equipment used to 'squeeze' it into position.

In some geological environments these requirements for slope modification may necessitate virtually the whole of the cutoff area being excavated or covered with concrete, e.g. Figure 17.4. It should be noted that in some cases there will be tendency for the rock to pluck out on joints, repeating the over-steepening problem. In these cases backfill concrete, or possibly pre-splitting with light blasting, may be necessary.

Large scale slope modification such as that shown in Figure 17.11 to avoid differential settlement and resultant cracking of the earthfill core has been used on many dams, e.g. Pratt *et al.* (1972) describe slope modification for Mica and Bennett Dams in Canada. The work carried out is shown in Figures 17.14. Walker and Bock (1972) give details of slope correction work at Blue Mesa Dam (Figure 17.15). In this case some of the slope correction work was required to remove loose unstable rock from the abutments.

Since these projects were constructed it has been recognized that earthfill core for dams may crack even in ideal conditions and that the best line of defence is to provide good filters (see Chapter 8). Hence the desirability of such major slope correction works is less clear. Slope correction as shown in Figure 17.11 would reduce the likelihood of wide deep cracks forming due to cross valley differential settlement, but not eliminate the possibility of cracking.

The authors' assessment is that provided the overall slope is less than 0.25H:1V (preferably 0.5H:1V) and good filters are provided, one should not be too concerned about the large scale slope modification carried out in the examples above.

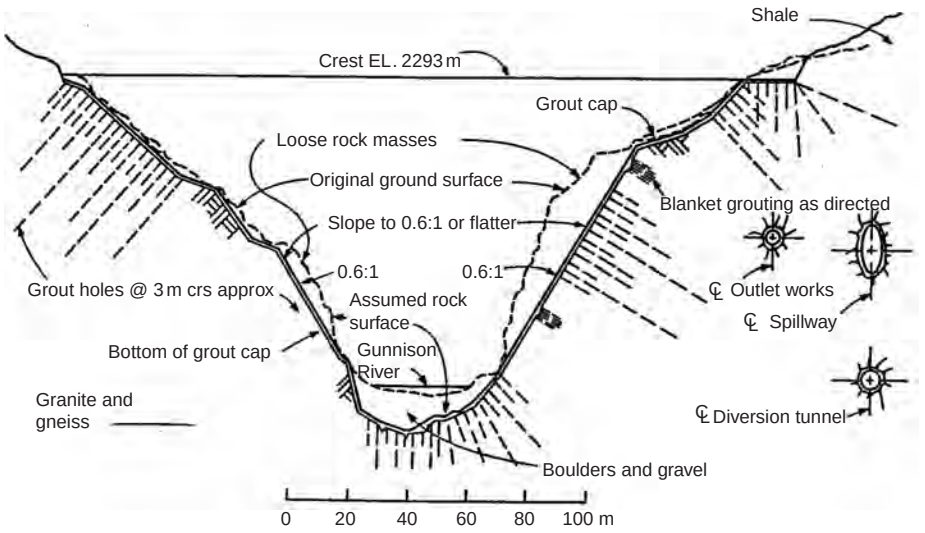


Figure 17.15 Foundation slope correction, Blue Mesa Dam (reproduced from Thomas, 1976). Reprinted by permission of John Wiley and Sons Inc.

A more conservative approach to large scale modification might apply for dams in severe earthquake areas, where differential movements and cracking may be more likely, and for very large dams. However even then the real line of defence is to provide good filters to control erosion which may initiate in cracks.

17.6.2 Seam treatment

It is common practice to require that seams of clay or extremely weathered rock which occur in the cutoff foundation should be excavated and filled with concrete. This is done to avoid erosion of the seams thus allowing seepage to bypass the earth core and filters.

Thomas (1976) suggests that the depth of excavation and backfill should be 2 to 3 times the width of the seam. USBR (1984) recommends that openings narrower than 50 mm should be cleaned to a depth of three times the opening width, and seams wider than 50 mm and up to 1.5 m should be cleaned to a depth of three times the width of the opening or to a depth where the seam is 12 mm wide or less, but not greater than 1.5 m depth. Wallace and Hilton (1972) indicate that for Talbingo dam all seams wider than 12 mm were excavated to a depth equal to the width and filled with concrete, or that the area was covered with thick grout, pneumatically applied mortar or concrete. This latter approach seems more reasonable for narrow seams particularly over areas where there are a number of narrow seams.

Figure 17.16 shows seam treatment adopted for Kangaroo Creek Dam, and Figure 17.5 that adopted for Dartmouth Dam. For Thomson dam, the foundations of which was often closely jointed rock criss-crossed by seams, and crushed zones, the whole

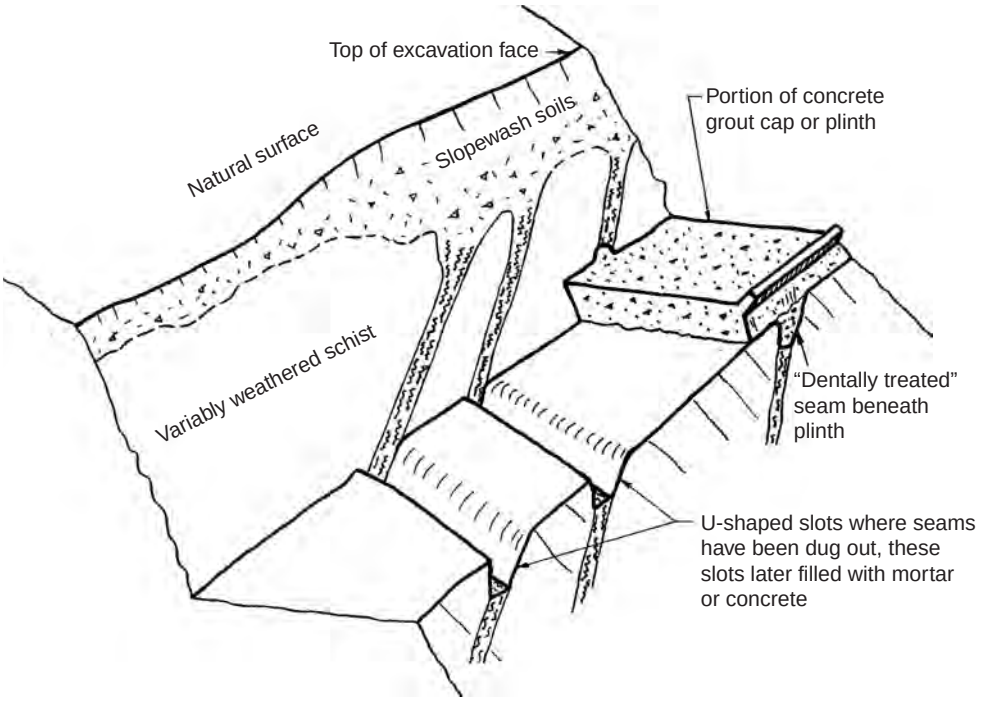

Top of excavation face
Natural surface
Slopewash soils
Portion of concrete grout cap or plinth
Variably weathered schist
"Dentally treated" seam beneath plinth
U-shaped slots where seams have been dug out, these slots later filled with mortar or concrete

Figure 17.16 Dental treatment of weak seams in the plinth foundation of Kangaroo Creek Dam.

of the cutoff was shotcreted except when it was already covered by slope correction concrete or backfill concrete for large seam treatment.

These examples can be used as a guide to reasonable practice. The authors would prefer not to dig out the seams, but to ensure that they are adequately covered with concrete or shotcrete, since their nature is not likely to change with one or two widths, and the concrete or shotcrete will be held in place by the earthfill.

For foundations with continuous seams filled with dispersive or erodible soil and rock, a horizontal filter drain may be needed downstream of the earthfill core to allow foundation seepage to emerge in a controlled manner.

17.6.3 Dental concrete, pneumatically applied mortar, and slush concrete

The following recommendations on the placement and quality of dental concrete, pneumatically applied mortar or concrete and slush concrete are taken from USBR (1984). They are consistent with the authors' experience of what constitutes good practice.

a) *Dental concrete*

Dental concrete is used to fill irregularities in the foundation due to joints, bedding, sheared zones, overhangs, or excavated surfaces.

Slabs of dental concrete should have a minimum thickness or 150 mm. Thin areas of dental concrete over rock projections in a jagged rock surface are likely places for concrete cracking and should be avoided by using a sufficient thickness of dental concrete or by avoiding continuous slabs of concrete over areas containing numerous irregularities. Feathering at the end of slabs should not be permitted, and the edges of slabs should be sloped no flatter than 45°. When formed dental concrete is required, it should not be placed at slopes greater than 0.5:1 (H:V), as discussed in Section 17.6.1.

When fillets of dental concrete are placed against vertical or near vertical surfaces, feathering should not be permitted, and a bevelled surface with a minimum thickness of 150 mm will be required at the top of the fillet.

The mix proportions should provide strength at 28 days of 20 MPa. The maximum aggregate size should not be larger than one-third the depth of slabs or one-fifth the narrowest dimension between the side of a form and the rock surface.

Aggregate and water shall be of quality equal to that required in concrete specifications allowing for sulphates in the foundation materials and groundwater. To ensure a bond between the concrete and the rock surface, the rock surface should be thoroughly cleaned and moistened prior to concrete placement. When overhangs are filled with dental concrete, it is essential that the concrete is well bonded to the upper surface of the overhang. Before concrete placement, the overhang should be shaped to allow air to escape during concrete placement and thus prevent air pockets between the concrete and the upper surface of the overhang. The concrete must be placed and allowed to set with the head of the concrete higher than the upper surface of the overhang. In cases where the preceding measures are not feasible, grout pipes should be inserted through the dental concrete to fill potential air voids. If grouting behind dental concrete is employed, grouting pressures should be closely controlled to ensure that jacking of the concrete does not occur.

Finished dental concrete horizontal mats should have a roughened, broomed finish (but not corrugated) to provide a satisfactory bonding surface to embankment material.

The dental concrete should be cured by water for at least 14 days or until the earthfill is placed over it (normally a minimum time of 72 hours is specified before the earthfill can be placed. The site engineers should be wary of using curing compounds as they act as bond-breakers; only compounds that can be removed or have broken down on exposure to air should be used. Care should be taken to ensure that cracking does not occur in the dental concrete due to subsequent earthfill placement and compaction operations. Earthfill operations may not be permitted over dental concrete for a minimum time interval of 72 hours or more after concrete placement to allow concrete time to develop sufficient strength to withstand stress caused by earthfill placement operations.

b) *Pneumatically applied mortar or concrete (shotcrete)*

Shotcrete is concrete or mortar that is applied pneumatically at high velocity with the force of the jet impacting on the surface serving to compact the concrete or mortar. Practice today is to use wet mixed shotcrete because it is easier to control. The quality of shotcrete is highly dependent upon the skill and experience of the

crew applying it, particularly with regard to their ability to prevent rebound from being entrapped in the shotcrete, control the thickness, prevent feather edges, and ensure adequate thickness over protrusions in irregular surfaces. There is also a danger of covering unprepared treatment areas because of the ease and rapidity of placement. Shotcrete should be used beneath impervious zones only when the use instead of dental concrete can be justified by site conditions. If it is used, the specifications should be very strict to ensure proper provisions for adequate quality work. The requirements for concrete quality, layer thickness, feathering and curing time should be as for dental concrete. The inclusion of reinforcement in shotcrete when used as a foundation surface treatment material for an earth-fill zone is not usually done. Light mesh to control early-age thermal/shrinkage cracking may be advisable for some jobs.

c) *Slush grout*

Slush grout is a neat cement grout or sand-cement slurry that is applied to cracks in the foundation. Slush grout should be used to fill only narrow surface cracks. It should not be used to cover exposed areas of the foundation, other than as described in Section 17.3.2 where it is used as a temporary cover over slaking or similar foundations.

Slush grout may consist of cement and water, or sand, cement and water. To ensure adequate penetration of the crack, the maximum particle size in the slush grout mixture should be no greater than one-third the crack width. The consistency of the slush grout mix may vary from a very thin mix to mortar as required to penetrate the crack. The grout would be mixed with a mechanical or centrifugal mixer and the grout should be used within 30 minutes after mixing.

Cracks must be cleaned out and wetted prior to placement of slush grout. Slush grout may be applied by brooming over surfaces containing closely spaced cracks, or by troweling, pouring, rodding, or funneling into individual cracks. The requirements for cement and aggregates should be as for dental concrete.

17.6.4 The need for good records of foundation treatment

It is essential that there is a good record of foundation treatment so those assessing the safety of the dam in the future can be informed on what was done. This should include:

- Contour survey of the cutoff showing areas of slope modification.
- Mapping of the geology of the cutoff foundation.
- Plans showing the treatment of the cutoff which can be related to the geological mapping.
- Photographs of the cutoff clearly marked so they can be located.

Figures 17.17 and 17.18 are excellent examples of such records from WAC Bennett Dam in British Colombia. These plans were prepared for 100 feet (30 m) square sections of the foundation.

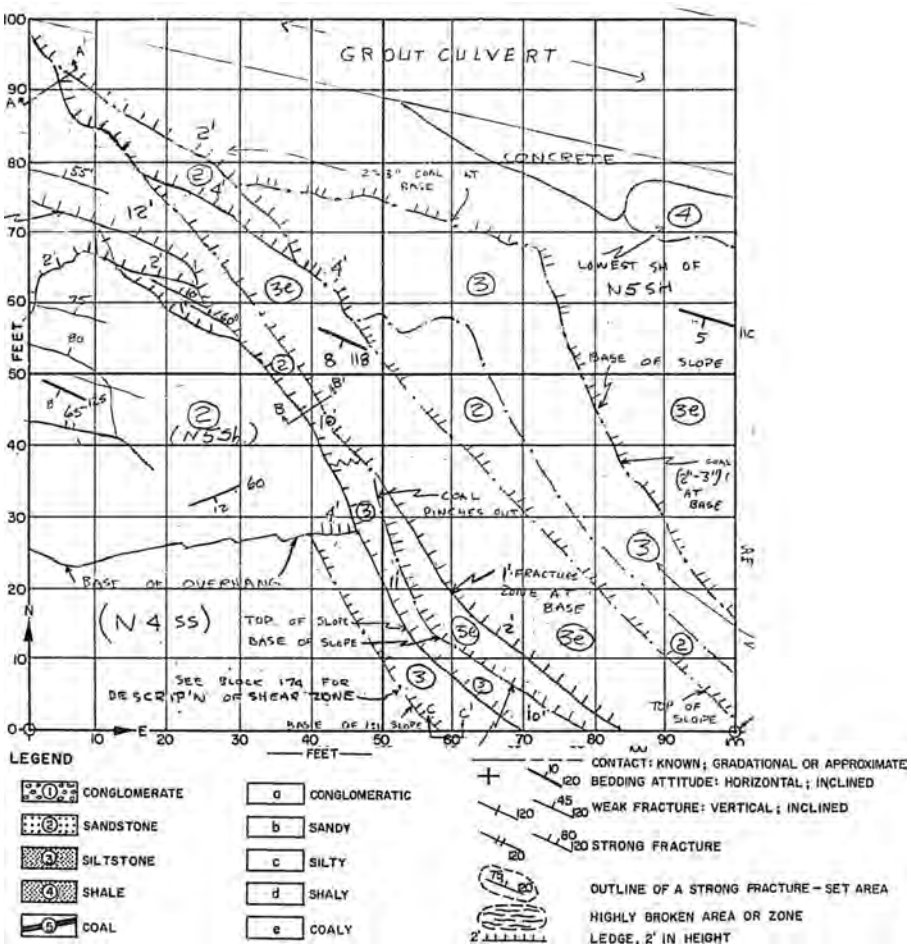


Figure 17.17 Example of mapping of foundation cutoff-WAC Bennett Dam, British Colombia (courtesy of BC Hydro).

17.7 ASSESSMENT OF EXISTING EMBANKMENT DAMS

When assessing the likelihood of internal erosion and piping of existing dams, a thorough assessment should be made of what foundation preparation was carried out. This can be assessed from construction reports, specifications and most particularly from photographs taken during construction. While it was common practice to carefully clean up (of loose material) the cutoff foundation in earlier dam construction, it was not until around the 1960s to 1970s that extensive use was made of dental concrete. Hence many dams built before then have the potential for loose/softened zones of core material near the contact between embankment and foundation. These have been detected in some dams using cone penetration tests through the earthfill.

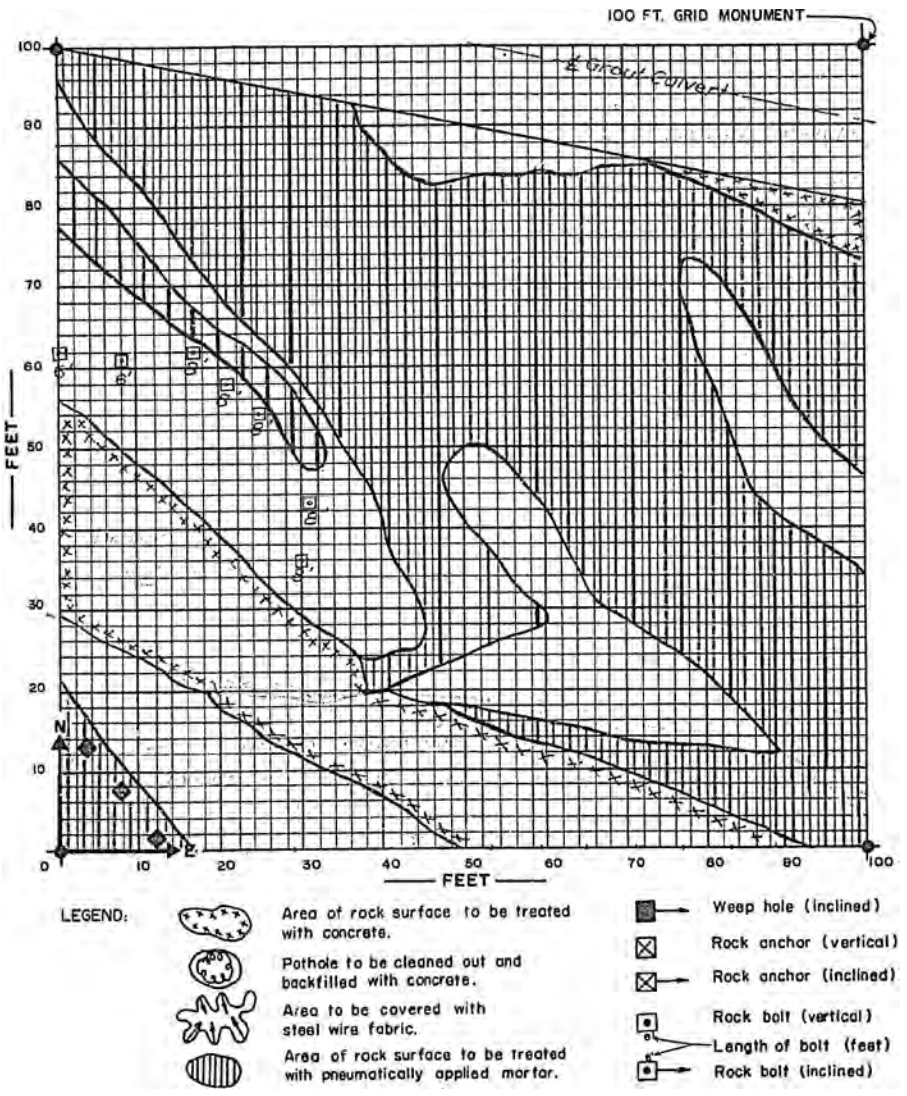

100 FT. GRID MONUMENT
FEET
FEET
LEGEND:
Area of rock surface to be treated with concrete.
Pothole to be cleaned out and backfilled with concrete.
Area to be covered with steel wire fabric.
Area of rock surface to be treated with pneumatically applied mortar.
Weep hole (inclined)
Rock anchor (vertical)
Rock anchor (inclined)
Rock bolt (vertical)
Length of bolt (feet)
Rock bolt (inclined)

Figure 17.18 Example of record of foundation cutoff treatment-WAC Bennett Dam, British Colombia (courtesy of BC Hydro).

As discussed in Chapter 8 large scale irregularities in the foundation have the ability to lead to low stresses near the crest of the dam. Often the profile is recorded on the drawings showing the results of grouting and/or on photographs, but just as often they are not. On a number of projects construction of haul roads or access tracks across the core foundation has resulted in benching followed by differential settlement and cracking of the core. Whether these benches are of concern, often relates to how continuous they are through the core of the dam. If they are through the whole core, they are of greater concern than if they only persist for say 5% of the width of the core.

17.8 FOUNDATION PREPARATION FOR CONCRETE GRAVITY DAMS ON ROCK FOUNDATIONS

17.8.1 The general requirements

a) *Foundation strength*

The primary critical issue is whether there are continuous, or near continuous, weak, unfavourably oriented discontinuities in the foundation e.g.:

- bedding surfaces
- bedding surface shears
- stress relief (sheet) joints
- faults and shears.

The geotechnical investigations must assess the foundation for such features and either the dam is designed to found on them or excavation goes below the feature.

The secondary but important issue is that the foundation must be cleaned of loose and loosened rock, soil, water and other matter, prior to placing the concrete so that there is a good bond between the foundation and concrete.

b) *Foundation modulus*

The requirement is that the rock mass has a high enough modulus so deflections under the loads of the dam are not excessive. The rock mass modulus is related to the rock substance strength, but in particular to the nature of the discontinuities, including spacing, opening, degree of infilling and orientation relative to the load direction.

Consolidation grouting (see Chapter 18) may be used to grout the joints and other fractures to reduce permanent deformations under the load of the dam by filling the open defects if it is not economic to remove this material. The grouting is however more likely to make the rock mass more an elastic medium, than to necessarily greatly increase the modulus.

c) *Erodibility and uplift pressure*

The seepage gradients beneath concrete dams, particularly arch dams, can be high, so it is necessary to carefully consider the erodibility of the joints, particularly if the joints are infilled with clay. Foundations would usually be excavated to non-erodible rock, and the rock grouted with consolidation (and curtain) grouting to reduce the flow of water through the rock. Borehole drains are provided to intercept the seepage water and reduce the uplift pressures under the dam and its foundation. The design of these drains is discussed in Chapter 16.

17.8.2 Excavation to expose a suitable rock foundation

The upper surface of rock which can be treated economically to meet these requirements is predicted from the results of the site exploration, as presented in the geotechnical model. The predicted surface is shown on drawings by means of cross sections and/or contours. Invariably, some excavation is required to remove overlying materials which may include the following:

- Soils – colluvial, alluvial and residual.
- *In situ* rock mass, the substance and defects of which are too weak and compressible, either inherently or as a result of weathering.



Figure 17.19 Middle River Dam – foundation cleanup in progress.

– Mechanically loosened *in situ* rock mass with clay-infilled joints. This may be adequately strong in substance but the mass is too compressible or weak as it contains too many seams which cannot be treated effectively by jetting and grouting.

The excavation is aimed at reaching the required foundation surface, while causing minimal damage to the rock below it. Some drilling and blasting may be required, but usually the rock next to the surface is removed by rock breaker, backhoe and hand methods. The surface reached in this way is cleaned up by compressed air/water jetting, with further use of hand tools. The cleaned-up surface should contain no detached rock fragments or loosened or drummy blocks and details of the exposed rock substances and defects should be clearly visible. Figure 17.19 shows cleanup in progress at Middle River Dam, a concrete gravity dam on Kangaroo Island, South Australia.

The cleaned-up surface should be mapped geotechnically in detail. Figure 17.20 is a sample of such mapping at Sturt River Dam in South Australia. The purpose of this mapping is to:

– Confirm that the foundation surface rock is of the required and predicted quality.
– Confirm that the rock structure is essentially as predicted in the geotechnical model and allowed for in the design and, in particular, that there is no unexpected geological situation, e.g. a kinematically feasible failure surface, which may require a change to the design.

The mapping is preferably done while the cleanup is in progress, as this can allow special attention to cleanup of important defects and timely decisions to be made, e.g. on the need for dental treatment, further deepening or design changes.

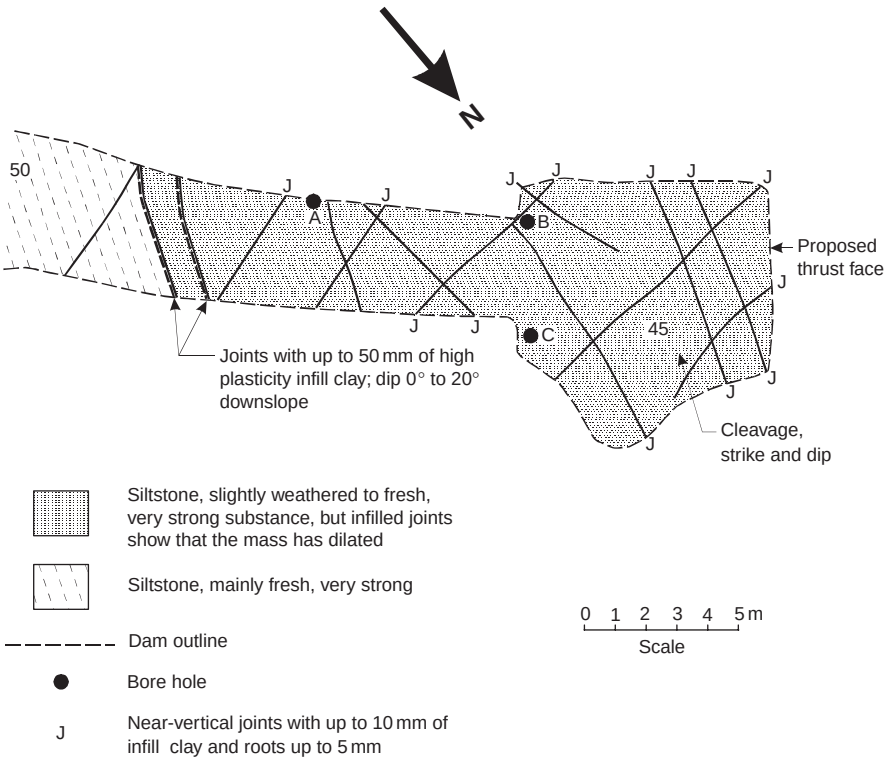

N
50
A
B
C
45
J
Proposed thrust face
Joints with up to 50 mm of high plasticity infill clay; dip 0° to 20° downslope
Cleavage, strike and dip
Siltstone, slightly weathered to fresh, very strong substance, but infilled joints show that the mass has dilated
Siltstone, mainly fresh, very strong
Dam outline
Bore hole
Near-vertical joints with up to 10 mm of infill clay and roots up to 5 mm
0 1 2 3 4 5 m
Scale

Figure 17.20 Sturt River Dam, upper right bank and thrust block, part of log of foundation at initial design level.

17.8.3 Treatment of particular features

a) *Dental treatment of localised weak/compressible features extending into acceptable foundation rock.*

Features for which localized dental treatment is suitable may range from less than 10 mm to several metres in thickness and include faults (sheared and/or crushed zones) and zones of altered or weathered rock. When steeply dipping, they may extend well below the otherwise acceptable foundation surface. If it can be shown that they do not combine with other defects to form kinematically feasible blocks or wedges, such features may simply represent:

- Local zones of possible under-seepage through potentially erodible materials and
- If wide enough, local zones of unacceptably high compressibility.

In such cases they can be treated locally by excavation and backfilling with mortar or concrete. Figure 17.21, based on Nicol (1964), shows this form of treatment applied to a fault (2 m to 6 m thick sheared, partly crushed zone)

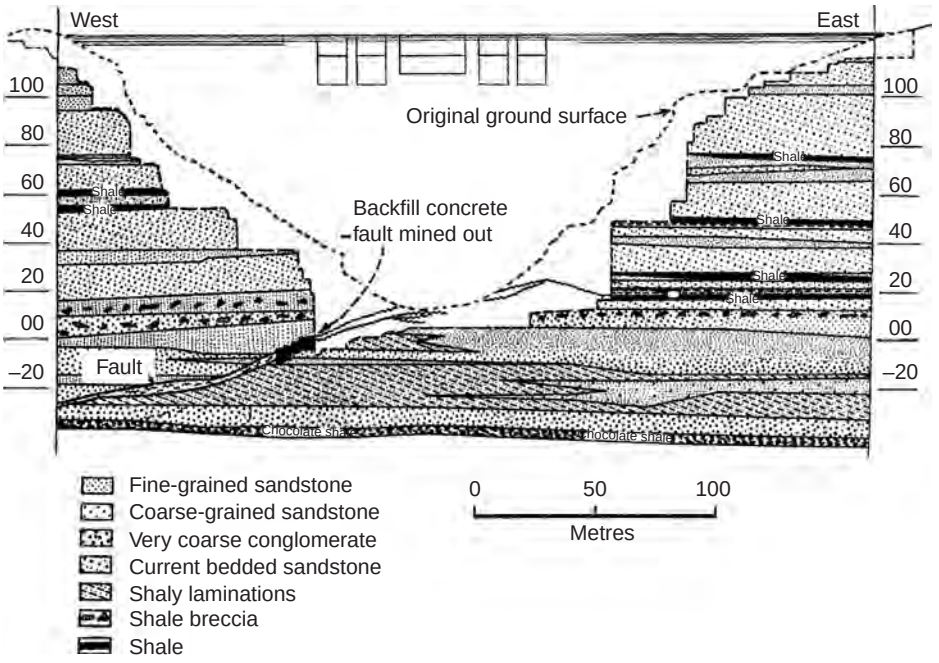


Figure 17.21 Backfill concrete of a fault at Warragamba dam (based on Nicol, 1964).

which passed beneath the valley floor and left abutment at the 137 m high Warragamba Dam, near Sydney. In the valley floor the dam excavation was taken down to below the zone. Beneath the left abutment the zone was mined back for about 17 m from the main excavation and backfilled with concrete. In the right abutment the fault did not appear as a major defect; it may have been represented by polished surfaces in a shale unit.

b) *Rock found to contain weak defects which provide kinematically feasible failure surfaces.*

As discussed in Section 17.8.1 rock containing weak defects which provide feasible failure surfaces should not normally occur at or below the design foundation surface. When such defects are found during foundation excavation, the design must be re-examined and, if necessary, modified. The following are examples of this and illustrate some types of treatment which have been applied.

Sturt River Dam (South Australia)

This 40 m high multiple curvature arch dam (Figures 17.22 and 17.23) was built during 1964–66 as a flood control structure. It is located in a steep rocky gorge, formed by siltstone which is very strong when fresh. The rock has well developed cleavage, dipping around 50 degrees upstream. Bedding is rarely visible. Planning stage core drilling showed the rock to be variably weathered in the abutments and the design allowed for excavation of the weathered material to produce a foundation on mainly

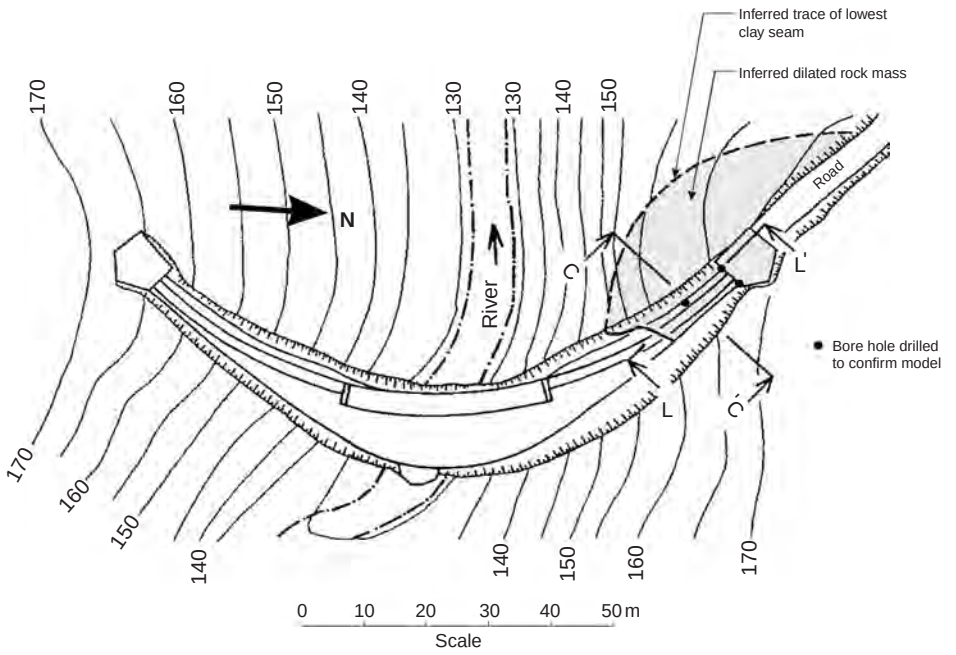


Figure 17.22 Sturt Dam – plan showing inferred displaced wedge in upper right bank (courtesy SA Water).

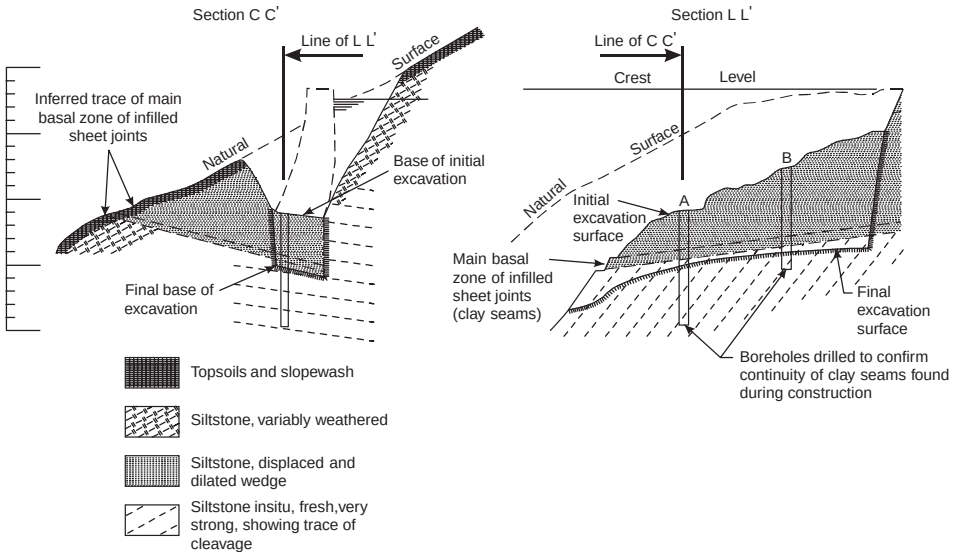


Figure 17.23 Sturt Dam – sections through displaced wedge at the top of right bank (courtesy SA Water).

fresh rock, containing few joints. The design assumed 6–10 m of excavation in the upper right abutment (Figure 17.23). Figure 17.20 shows the geotechnical log of this part of the foundation when the excavation reached the design levels. The mainly fresh rock exposed here showed a pattern of near-vertical joints, most of which were infilled with up to 10 mm of high plasticity clay. Down slope from these joints, the traces were found of 2 seams up to 50 mm thick of similar clay, dipping gently and obliquely down slope and upstream. From this evidence it was inferred that the gently dipping seams were infilled sheet joints. It was further inferred that the whole rock mass above the lower gently dipping seam was a mechanically loosened wedge or rock, which had dilated and moved slightly down dip along these seams. The gently dipping seams were judged to represent kinematically feasible failure surfaces. The extrapolated trace of the lower seam (Figure 17.20) coincided with minor depressions at the ground surface. This inferred model was confirmed by high quality core drilling (Figures 17.22 and 17.23), which recovered the predicted seams and showed fresh siltstone with few tightly closed joints below the lower seam.

To reach this undisturbed fresh rock, the foundation was deepened locally by 4 to 7.5 m. Immediately after the deepening, monitored cracks and displaced traces of presplit holes indicated that the isolated downstream part of the wedge had dilated further, and was creeping down dip. It was braced immediately by a concrete buttress placed on the undisturbed rock and extending across to the upstream side of the excavation. The final design provided extra support to this upper part of the dam by means of prestressed anchors installed through it into the foundation.

Clyde Dam (New Zealand)

This gravity dam of maximum height 100 m is located on schist, about 3 km from the Dunstan Fault, which is believed to be active (Hatton and Foster, 1987, Paterson *et al.*, 1983). At the dam a steeply dipping fault parallel to the river passes upstream-downstream through the foundation. This River Channel Fault comprises up to 8 m of crushed material and is believed to be a normal fault with some horizontal component. Seismotectonic studies carried out during the dam construction indicated that up to 200 mm of displacement might be induced across this fault under the dam, if the Dunstan Fault suffered a major rupture. The dam design was modified by the inclusion of a slip joint above the fault, as shown on Figure 17.24. The joint is designed to accommodate up to 1 m of dip-slip movement and up to 2 m of strike-slip movement. The design is described by Hatton and Foster (1987) and Hatton *et al.* (1991).

The foundation contains many other faults, mostly sheared, partly crushed zones. Some (locally termed foliation shears) occur parallel to the gently dipping foliation, and others cut across the foliation at steeper angles. Mapping of excavated foundations and exploratory drives during construction showed that some of the foliation shears could provide potential failure surfaces (Hatton *et al.*, 1991, Paterson *et al.*, 1983). These were treated either by local over-excavation and deepening of concrete structures (Figure 17.25) or by concrete shear keys placed in adits (Figure 17.26). Foliation shears with down slope components were exposed in the right abutment excavation for the dam. Some of these contained sandy infill of alluvial origin. Dilation of the rock mass due to past down slope creep was inferred. Four of these shears were explored by adits which were later backfilled with concrete to act as cutoffs.

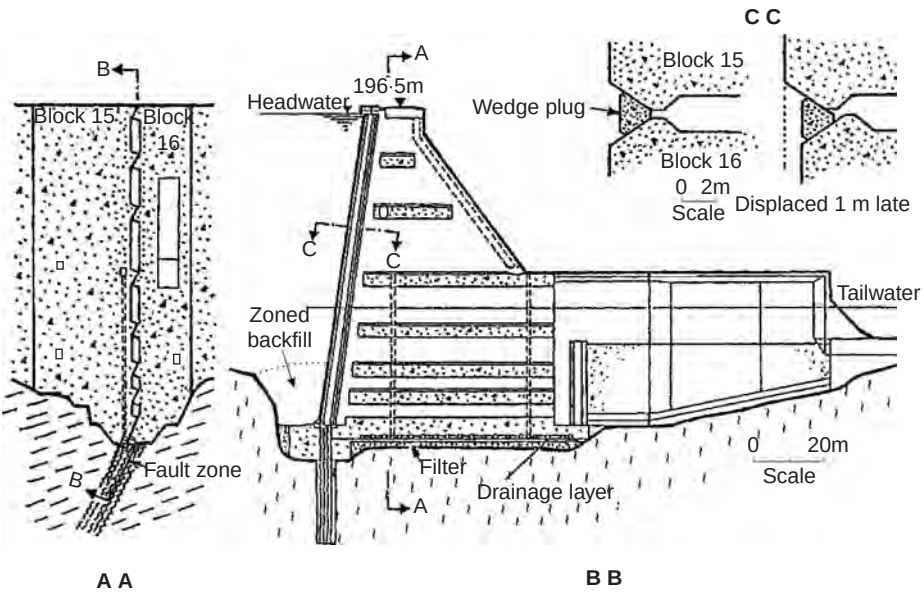


Figure 17.24 Clyde Dam showing design of slip joint.

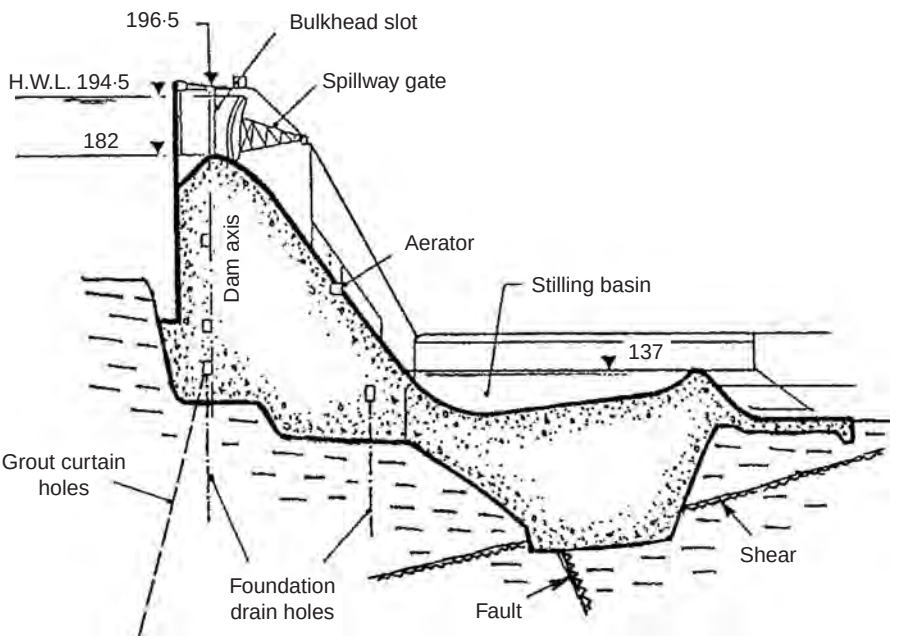


Figure 17.25 Clyde Dam showing local deepening of the concrete to found below a shear surface.

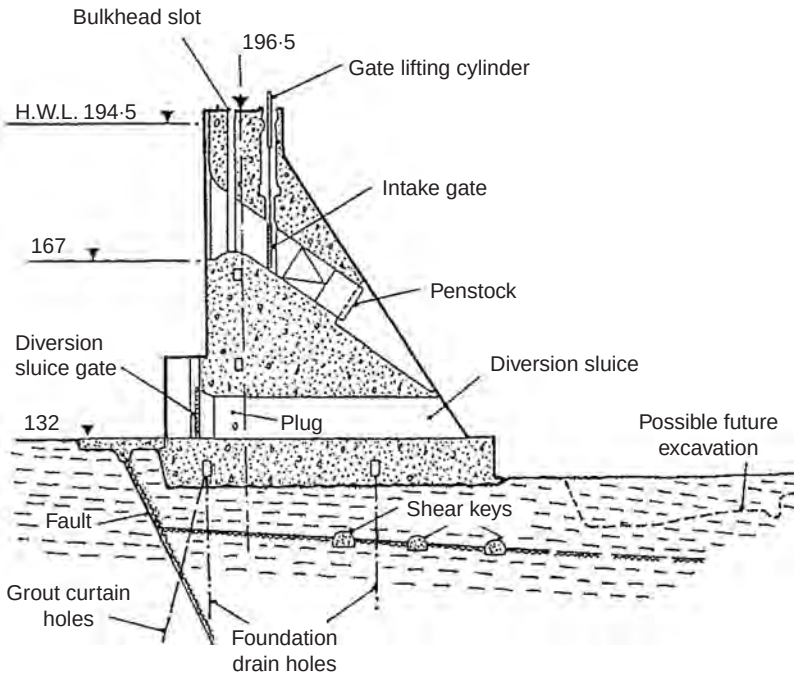


Figure 17.26 Clyde Dam showing concrete shear keys in a shear surface to increase shear strength.

17.8.4 Treatment at sites formed by highly stressed rock

Excavation to reach foundation level in highly stressed rock should be done bearing in mind the following:

- Exposed highly stressed rock invariably has sheet joints as its surface and a succession of sheet joints below (see Figure 2.5).
- Any sheet joint below the base of a concrete dam has some potential to form all or part of a downstream sliding surface.
- When excavations are made into highly stressed rock, it is possible that some existing sheet joints may open up further and propagate, and some new sheet joints may form (see Section 2.5.4). Such effects are much more likely to occur if blasting is used.

At Burdekin Falls Dam in Queensland, the approaches to selection of foundation levels and to excavation methods were developed with understanding of the above issues. The following account is based on Lawson and Burton (1990), Lawson *et al.* (1992) and Russo *et al.* (1985).

The dam is a concrete gravity structure 38 m high and 876 m long. Most of its length comprises 34 monoliths which mainly across the 600 m wide river bed. The site is formed almost entirely by welded tuff, which is fresh and extremely strong. Near-horizontal sheet joints are exposed over much of the river bed (Figure 17.27), and

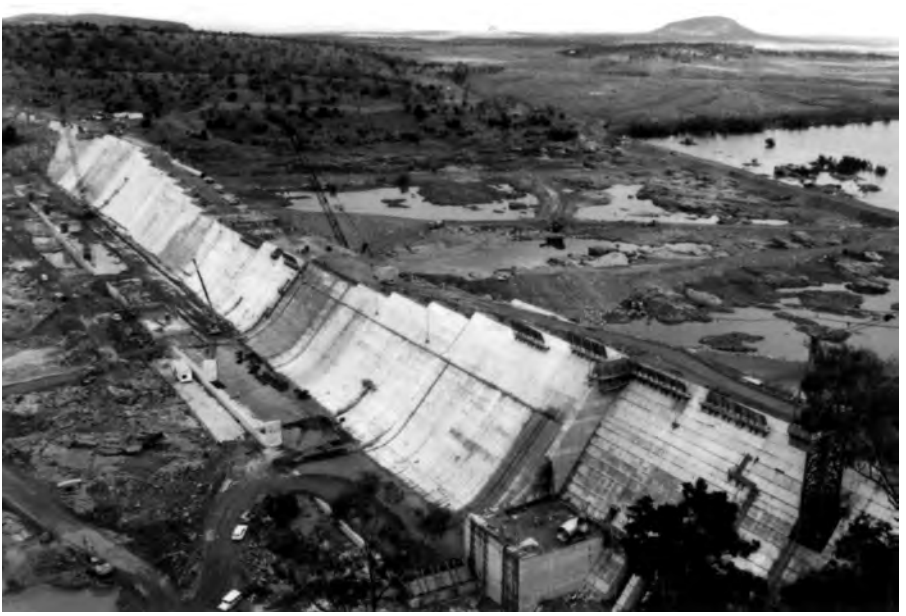


Figure 17.27 Burdekin Falls Dam, under construction, showing the 600 m. wide valley floor, formed by extremely strong, sheet-jointed rock (photograph courtesy of SunWater).

dipping sheet joints occur in both abutments. Planning stage investigations provided general understanding of the spacing, depths and persistence of the sheet joints, and showed that some were partly infilled with clay, usually of alluvial origin. Tests showed the joints to have very high shear strengths, and that only a few square metres of intact rock per monolith would be required, to meet established criteria for safety against sliding. High stresses were predicted from the sheeted structure, discing of drill cores and excessive propagation of fractures during and after blasting. Bock *et al.* (1987) showed the maximum principal stress to be horizontal, striking 10° to 30° to the dam axis. Values ranged generally from 10 to 20 MPa across the river bed, rising locally to 30 to 35 MPa at the left abutment.

During construction, the depth to adequate foundation rock was established progressively during the excavation for each monolith. In the river bed, excavation was mainly by rock breakers and excavators. Explosives were used for some slabs which were too large to handle. Cleanup was by bobcat, hand shovels and hosing. The adopted foundation surfaces ranged generally from about 1 down to 4 metres below the river bed. They were chosen for each monolith from the results of geological inspection of progressively exposed surfaces, and by the drilling, cleaning out, and down-hole inspection of at least 3 vertical holes, each 2.8 m deep. An Olympus IF6D2-30 Fiberscope was used for these inspections. A simple log for each hole recorded the depth of each sheet joint present and its aperture or infilling. Based on these logs, judgements were made on the persistence and character of the joints, and decisions made on the



Figure 17.28 Slab of rock which has lifted about 50 mm due to stress relief. Monolith LHZ adjoining to LH1 (photograph courtesy of SunWater).

need for either removal of the rock above them, or grouting. Extra holes were sometimes needed. The final (adopted) cleaned up surfaces were mapped geologically in plan view on 1:200 scale. The as-constructed record includes these plans and the logs of the 2.8 m holes.

In the abutments, blasting was used for much of the excavation, but smooth blasting techniques were specified. The as constructed records include foundation geology mapped on 1:100 scale, plus logs of the 2.8 m holes.

Significant stress-relief effects during the foundation preparation occurred only close to the base of each abutment, during hot weather. They included rock bursting (spalling of small, thin slices or rock), and slabbing (buckling up of slabs up to 100 mm thick and many square metres in area). Figure 17.28 shows an example of this.

Chapter 18

Foundation grouting

18.1 GENERAL CONCEPTS OF GROUTING DAM FOUNDATIONS

The foundations for most dams more than 15 m high built on rock and, for some which are smaller, are treated by grouting. Grouting consists of drilling a line or lines of holes from the cutoff level of the dam into the dam foundation and forcing cement slurry, or chemicals under pressure into the open defects in the rock, that is open joints, fractures, bedding partings and faults. Figure 18.1 shows an example:

The grouting is carried out to:

- Reduce leakage through the dam foundation, i.e. through the defects.
- Reduce seepage erosion potential.

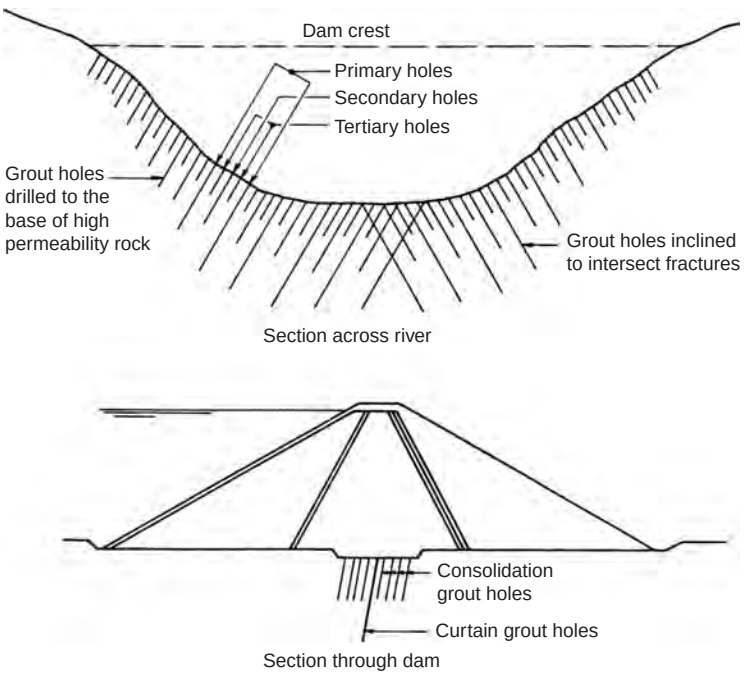


Figure 18.1 An example of embankment dam foundation grouting.

– Reduce uplift pressures (under concrete gravity dams when used in conjunction with drain holes).
– Reduce settlements in the foundation (for concrete gravity, buttress and arch dams).

Virtually all dam foundation grouting uses cement grout: Portland cement mixed with water in a high speed mixer to a water-cement ratio (mass water/mass cement) of between 0.5 and 5 to form a slurry which is readily pumped and able to penetrate defects in the rock in the dam foundation. Modern grouts include additives to maintain a stable grout (not subject to bleed) and to reduce the viscosity which improves penetration into open defects in the rock.

If the dam is on a soil foundation (e.g. sand) or if the fractures in the rock are very narrow, chemicals can be used instead of cement. Chemicals tend to be more expensive so are only used where cement grout would not be successful. The use of chemical grouting needs also to be chosen with care as some chemicals are potentially very hazardous materials, not only for the users, but also in the long term for people downstream from the dam. Most soil foundations are not grouted, because the soil acts as a filter to cement grout particles and the grout cannot penetrate into it. Control of seepage in soil foundations is more often achieved with cutoffs as detailed in Chapter 10.

Foundation grouting takes two forms:

– Curtain grouting.
– Consolidation grouting.

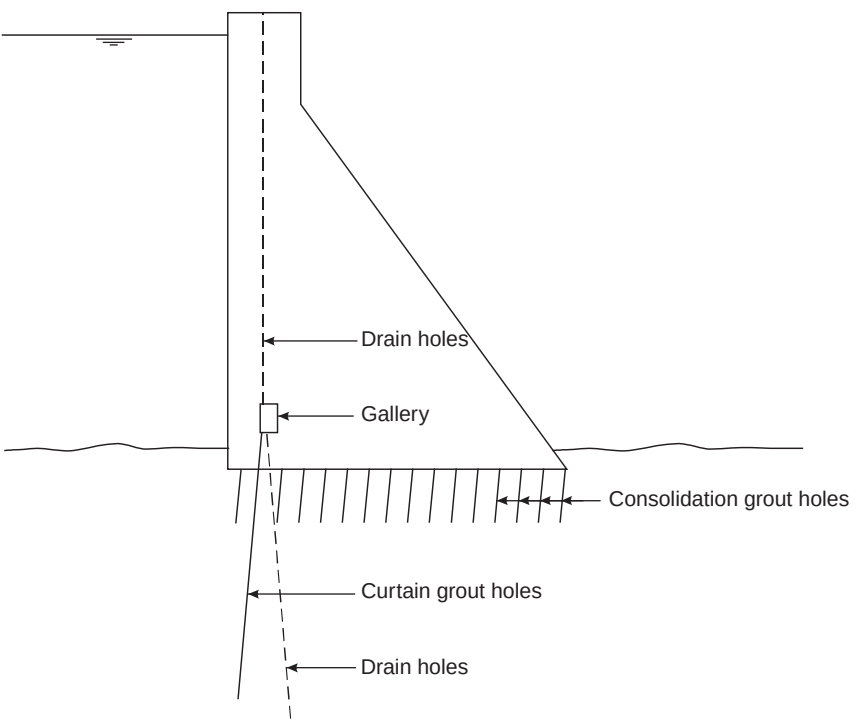


Figure 18.2 Curtain and consolidation grouting for concrete gravity dams.

Curtain grouting is designed to create a narrow barrier (or curtain) through an area of high permeability. It usually consists of a single row of grout holes which are drilled and grouted to the base of the permeable rock, or to such depths that acceptable hydraulic gradients are achieved. For large dams on rock foundations, dams on very permeable rock, dams where a high degree of confidence is required to achieve a good quality grout curtain or where grouting is carried out in soil foundations, 3, 5 or even more lines of grout holes may be adopted. Multiple row curtains are also adopted if, as at Thomson dam, it was impracticable to excavate the foundation to below the limit of the infilled joints.

The holes are drilled and grouted in sequence to allow testing of the permeability of the foundation (by packer testing) before grouting and to allow a later check on the effectiveness of grouting from the amount of grout accepted by the foundation ('grout take'). Thus in Figure 18.1 primary holes are drilled first, followed by secondary and then tertiary. The final grout hole spacing will commonly be 1.5 m or 3 m, but may be as close as 0.5 m. This staged approach allows control over the amount and effectiveness of the grouting.

Grout pressures are usually (at least in Australia, USA and the United Kingdom, for example) limited to prevent hydraulic fracture of the rock. The discussion in this chapter is predicated on that assumption. In some projects particularly in continental Europe, the rock has been deliberately fractured to improve grout penetration.

Consolidation or 'blanket' grouting for embankment dams is designed to give intensive grouting of the upper layer of more fractured rock under all or part of the contact surface of an embankment core or a concrete dam, or in regions of 'high' hydraulic seepage gradient, e.g. under the plinth for a concrete face rockfill dam. It is usually restricted to the upper 5 m to 15 m and is carried out in sequence but commonly to a predetermined hole spacing and depth.

Figure 18.2 shows a section through a concrete gravity dam. The curtain grouting is located near the upstream face of the dam and usually carried out from a gallery in the dam; sometimes from the upstream heel of the dam. It is designed to reduce seepage through the foundations and, in conjunction with the borehole drain holes, to control uplift pressures.

18.2 GROUTING DESIGN – CEMENT GROUT

18.2.1 Staging of grouting

Grouting of holes is normally carried out in stages, the method depending on the permeability and quality of the rock being grouted and the degree to which control of the grouting operation is desired. Figures 18.3, 18.4, 18.5, 18.6 and 18.7 show the different methods available.

Downstage Without Packer (Figure 18.3): This is one of the preferred methods for high standard grouting, since each stage is drilled and grouted before the next, lower stage, allowing progressive assessment whether the hole has reached the desired closure requirement. This method allows higher pressures to be used for lower stages, as it reduces the risks of leakage from them up the top stage levels and gives better grout penetration between holes. The grout pressures are limited by the effectiveness

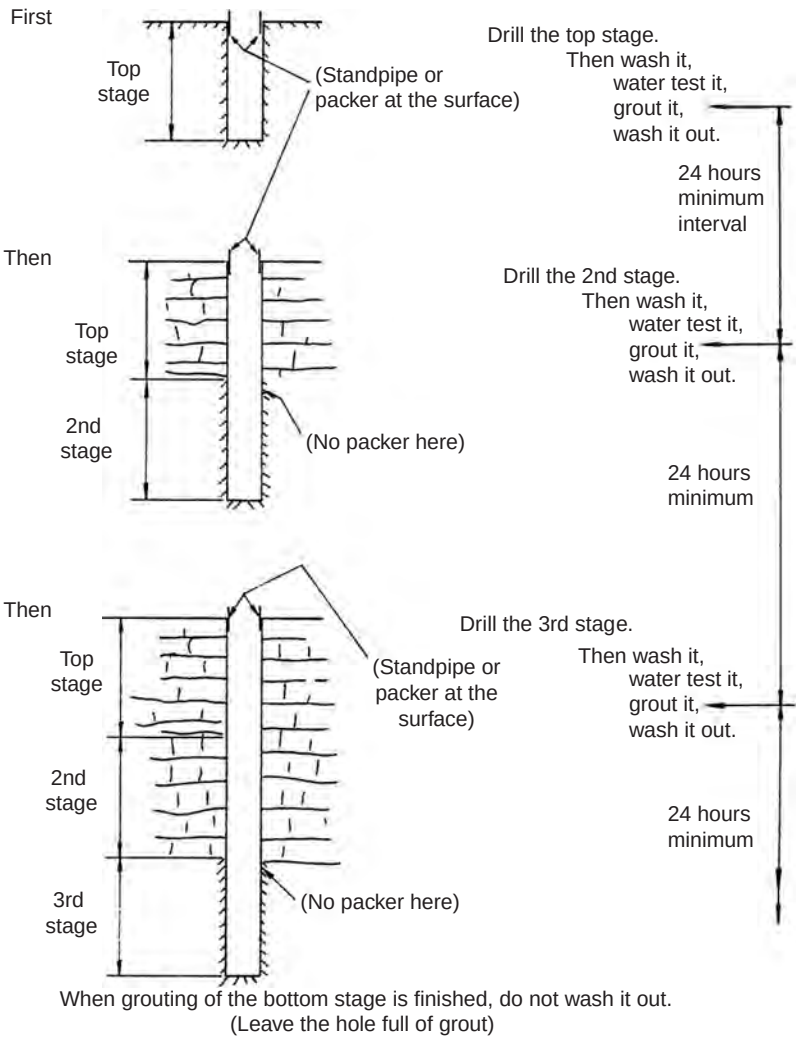


Figure 18.3 Grouting downstage without packer (WRC, 1981).

of the top stage grouting. It does necessitate a separate set up of the drill for each stage and separate 'hook-ups' of the grout lines. It is, therefore, relatively expensive. This method is the one preferred by Houlsby (1977, 1978, 1982a).

Downstage With Packer (Figure 18.4): This method allows use of increased grout pressures for lower stages, since these pressures are not applied from the surface. However, there may be problems with seating and leakage past the packer. Bleeding of the grout hole (i.e. removing the 'clear' water which accumulates at the top of the grout hole as the cement settles) cannot be achieved except at the ground surface (i.e. not immediately above the grout stage). Ewart (1985) indicates a preference for this

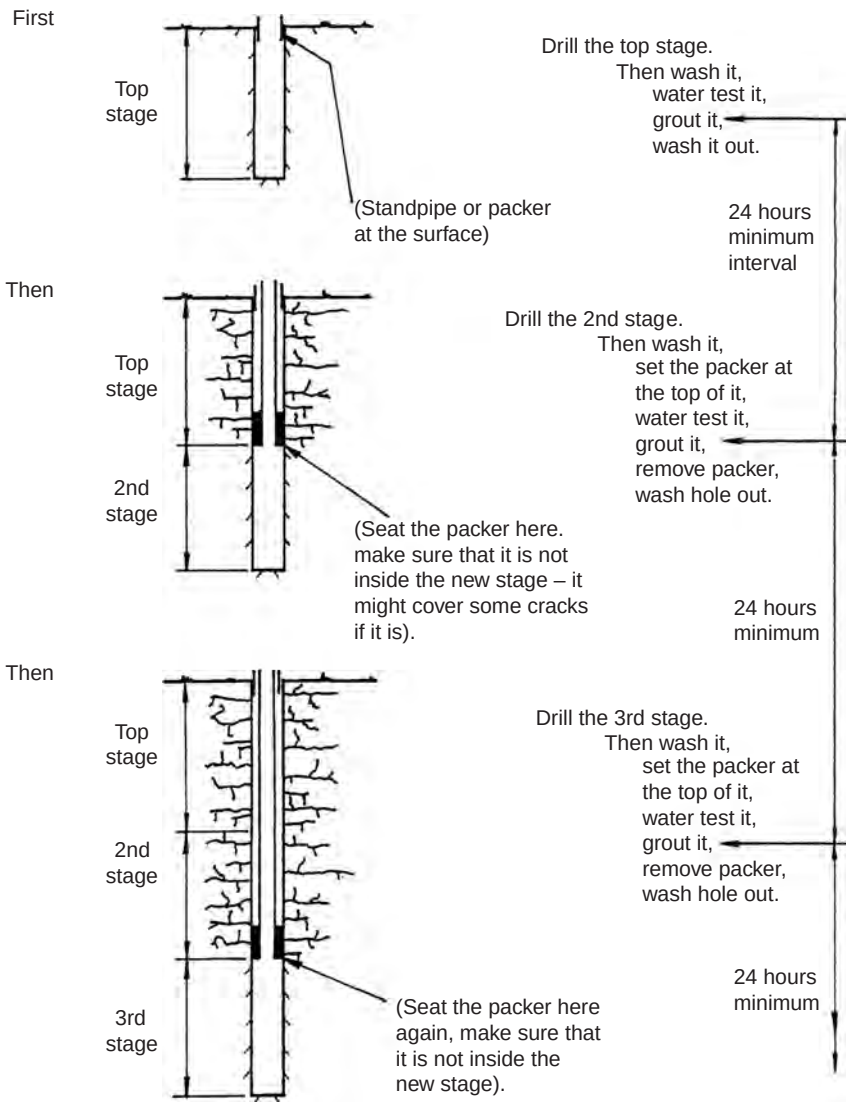


Figure 18.4 Grouting downstage with packer (WRC, 1981).

method, because of the potential to fracture the rock in the upper levels if downstage without packer methods are used.

Upstage (Figure 18.5): Does not allow progressive assessment of the depth of grout hole needed to reach a desired closure requirement as the holes are drilled to their full depth in one stage. The method is cheaper in principle than downstage methods since the drill rig is only set up once, but these savings may be offset by the need for more conservative total depths. The method is susceptible to problems with holes collapsing,

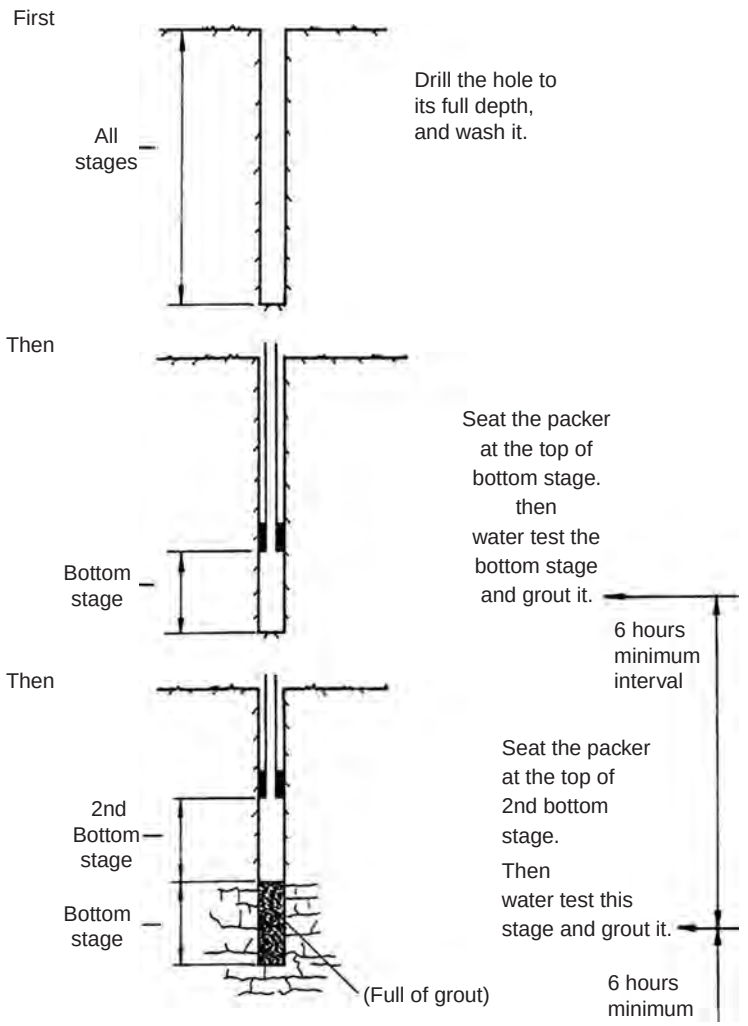

First
All stages
Drill the hole to its full depth, and wash it.
Then
Bottom stage
Seat the packer at the top of bottom stage. then water test the bottom stage and grout it.
6 hours minimum interval
Then
2nd Bottom stage
Bottom stage
(Full of grout)
Seat the packer at the top of 2nd bottom stage.
Then water test this stage and grout it.
6 hours minimum

Figure 18.5 Grouting upstage (WRC, 1981).

or over enlarging during drilling and grouting in poor rock conditions making seating of packer difficult or impossible. It is also subject to the same problems as the downstage with packer method regarding bleeding. The method is appropriate for secondary or tertiary holes, when depths are reasonably well known, and in strong rock with holes not likely to collapse or erode.

Full Depth (Figure 18.6): Does not allow proper assessment of where grout take is occurring or proper monitoring of reduction in Lugeon values with grouting. Depths are predetermined so the method does not allow logical assessment of grout depth based on closure requirements. Grouting pressures are limited. It is not an acceptable

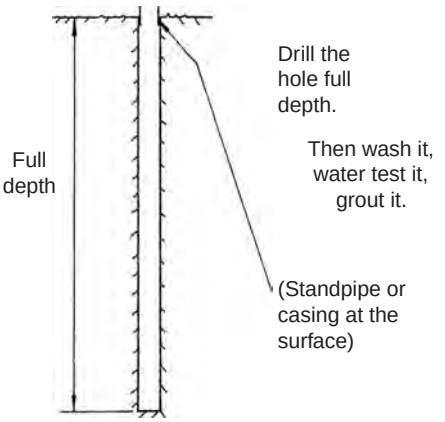


Figure 18.6 Grouting full depth (WRC, 1981).

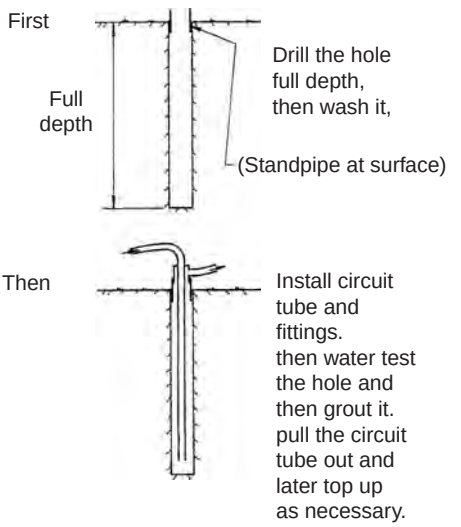


Figure 18.7 Grouting full depth, circuit (WRC, 1981).

method except for consolidation grout holes, probably where these holes are less than 5–8 m deep.

Full Depth Circuit (Figure 18.7): Has the same limitations as full depth grouting, but by injecting the grout into the base of the hole, the possibility of settled grout is lowered.

For stage grouting, the stage lengths are commonly predetermined depending on:

– The geological conditions and depths at which changes in degree of permeability are likely to occur.

- The minimum length worth drilling, since short stages are more costly to drill because of set up costs.
- Allowable pressures in the upper part of the hole (dependent on geological conditions).

Commonly grout stages will be 5 m to 8 m but may be increased in length lower in the foundation, e.g. Houlsby (1977) suggests:

Stage	*Depth Range (m)*
1	0 to 8
2	8 to 16
3	16 to 30
4	30 to 50

While this reduces the number of drill setups and grout hook-ups, it may result in unnecessarily deep holes, particularly for smaller dams.

Smaller than the predetermined stage lengths should be used when:

- Drilling water is lost, indicating a relatively large fracture or opening has been encountered or
- The grout hole is caving, due, for example, to closely fractured rock or
- Water flows into the hole under pressure or
- Very large water pressure test or grout takes are encountered (often it is possible to relate these to a specific geological feature) or
- For the top stage of a hole, when there is concern about the chance of over-pressuring the ground and cause heave, for example.

18.2.2 The principles of 'closure'

Apart from consolidation grouting, which may be carried out to a predetermined depth and hole spacing, grouting should be carried out sequentially to achieve a predetermined standard of water tightness. This will usually require the successive halving of hole spacing from primary to secondary to tertiary holes etc. as shown in Figure 18.1. Whether the required standard has been achieved will normally be determined on the basis of water pressure test Lugeon values on the grout stage prior to grouting and/or on grout take – the volume (or weight of cement) of grout per metre of grout hole. Closure criteria will be discussed in more detail below. Figure 18.8 shows the basic principles of hole closure, i.e. that secondary holes are drilled halfway between primary if water pressure tests (and/or grout takes) in the primary holes fail to meet the closure criteria.

Figure 18.9, taken from Houlsby (1977) and WRC, (1981), gives examples of the closure method when Lugeon water pressure test values are the required standard.

In these examples:

Case (a) Primary grouting has resulted in a reduction in Lugeon value (and grout take) in the secondary hole and tertiary grouting has resulted in further reductions close

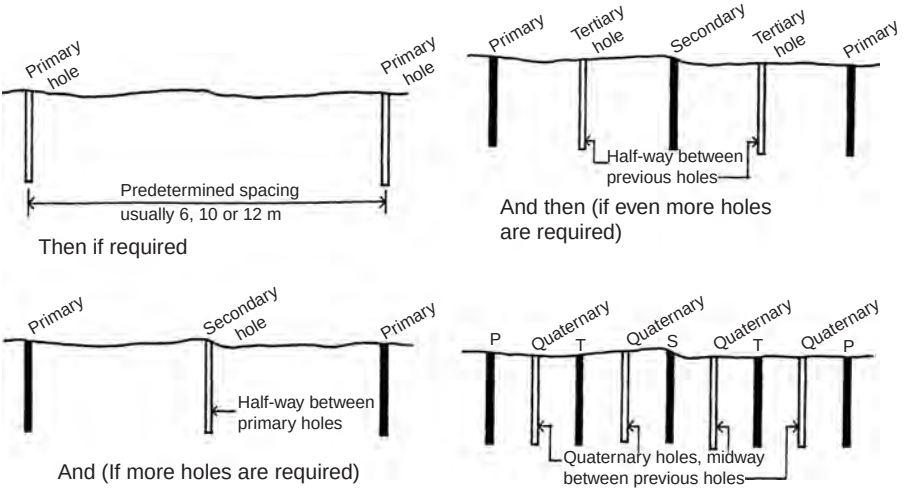


Figure 18.8 Basic concept of halving hole spacing to achieve closure.

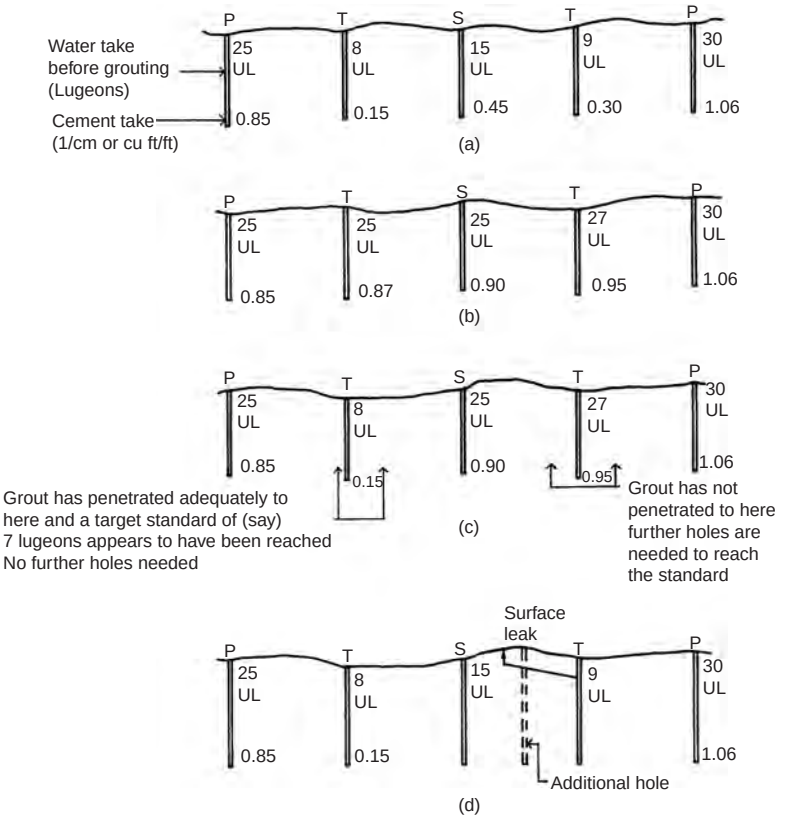


Figure 18.9 Examples of grout closure based on Lugeon water pressure test (and grout take as secondary information) adapted from Houlsby (1977) and WRC (1981).

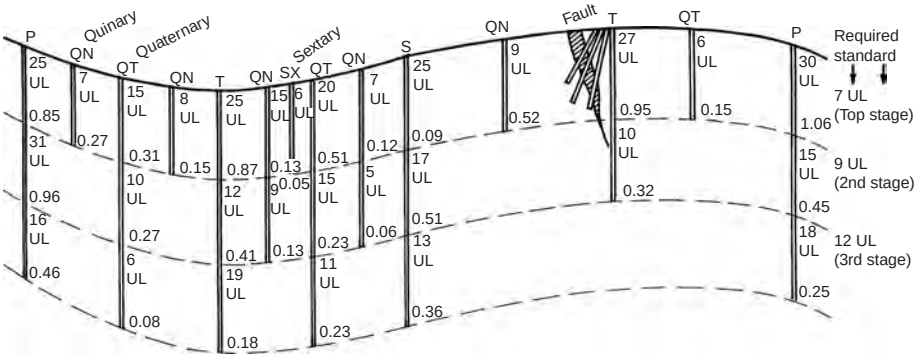


Figure 18.10 Example of closure based on water pressure test Lugeon criteria (adapted from Houlsby, 1977 and WRC, 1981).

enough to the closure requirement of 7 Lugeons (uL), hence no further grouting is required. If the closure requirement were say 5 uL, quaternary holes would have been required and would seem worthwhile as there is a progressive reduction in Lugeon value and in grout take.

Case (b) Primary and secondary grouting has resulted in no significant reduction in Lugeon value in the tertiary holes prior to grouting them. Houlsby (1977) concludes that quaternary holes are needed and perhaps quinary later. If holes are already at say 1.5 m spacing, the authors are of the view that consideration should also be given to whether grouting with cement is having any significant effect and whether grouting might be discontinued. In some cases for a curtain line, it could be better to opt for three lines of grouting, generally as described later in this section.

Case (c) A closure criterion of 7 uL has been achieved in part, but quaternary holes are required elsewhere. Since closure is being achieved in part it would seem viable to proceed to this quaternary stage of grouting where needed.

Case (d) As for Case (a) except that leakage to the surface occurred in grouting the tertiary hole, necessitating further grouting to seal the leak.

Closure requirements may vary with depth, and the hole spacing and depth required to achieve closure may also vary considerably. Figure 18.10 shows an example taken from WRC (1981). Note that this is a particularly thorough example of grouting, where if the primary spacing was 12 m the final spacing is 0.4 m in some areas. As discussed below, it is unlikely that closure to such a small hole spacing is warranted. This example also shows the use of "stitch grouting" to treat a known geological feature, in this case a permeable fault zone.

When more than one line of grout holes is planned, the holes should be drilled and grouted in sequence to form the outer lines ahead of the central line, so that the progressive development of closure can be observed. Figure 18.11 shows a possible closure sequence.

A deep 3-line curtain was completed successfully at Talbingo Dam, the 160 m high earth and rockfill dam in the Snowy Mountains Scheme. If the three lines are

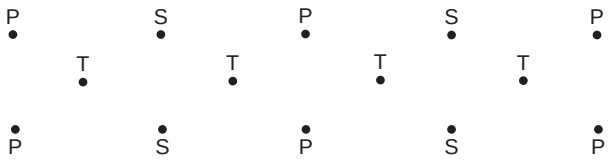


Figure 18.11 Grout closure with 3 lines of grout holes.

Table 18.1 Types of Portland cement and their application in grouting of dam foundations (adapted from Chaqui, 2006).

Australian Standard AS 3972	*ASTM Standard C 150*	*Description*
Type GP	Type I	Accepted as the general-purpose cement for the majority of grouting projects when the special properties of the other types are not required.
Type SR	Type II	Manufactured to resist moderate sulfate attack and to generate a slower rate of heat of hydration than Type I.
Type HE	Type III	Used when high early strength is required. It is considered for applications were fast sets are required. Also because it consists of finer particles it can be used to grout slightly smaller apertures than can be penetrated with Type I.
Type LH	Type IV	Generates less heat during hydration than Type II and develops strength at a slower rate than Type I. It can be used for applications were a large mass of grout will be placed and high hydration temperatures are unacceptable
n/a	Type V	Manufactured for use in grout exposed to severe sulphate action.
Microfine	Microfine	Has been ground finer to allow penetration of finer fissures or soil pores. These cements are available with a variety of different properties and may contain blast furnace slag as well as Portland cement.

designated A, B and C from downstream to upstream, the Talbingo procedure was to drill and grout the A-line holes to the specified depth, deep essentially as probe holes. The C-line holes, a slightly shallower line of holes, were drilled to their specified depth, but where indicated by high takes at depth, C-line holes were deepened as required. The B-line holes, also a specified shallower line, were finally drilled and grouted. If high takes persisted at depth in both the A and C holes, the appropriate B-line holes were deepened. With the B-line holes all 'contained' between the A- and C-lines, rarely were further closure holes needed. The total drilling (original and redrilling) requirement was greatly reduced and proved to be far easier to define. A similar procedure was used for Thomson Dam, also nearly 160 m high, where there were known open defects to up to depths of 90 m below natural surface and where nearly 8.5 times more cement was used for the grouting.

18.2.3 The design and quality control of cement grouts

18.2.3.1 *The cement and additives used for grouting*

Virtually all grouting of rock foundations for dams is carried out using cement based grouts. Table 18.1 summarizes the types of cement and their application.

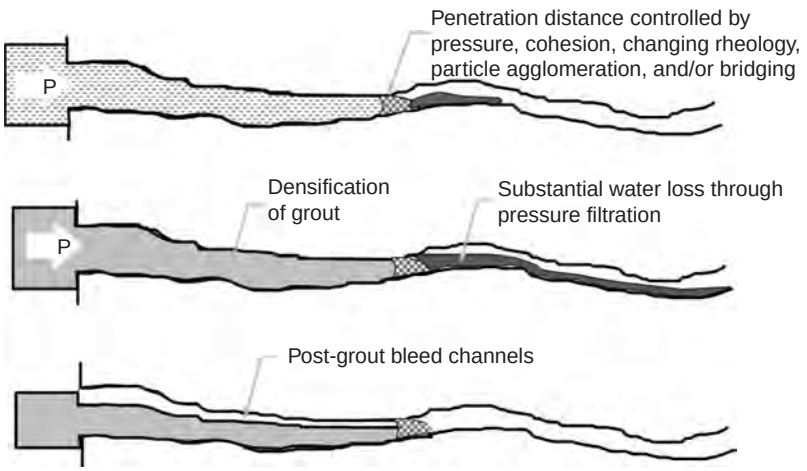


Figure 18.12 Performance of unstable grouts during and after injection (Stare *et al.*, 2013).

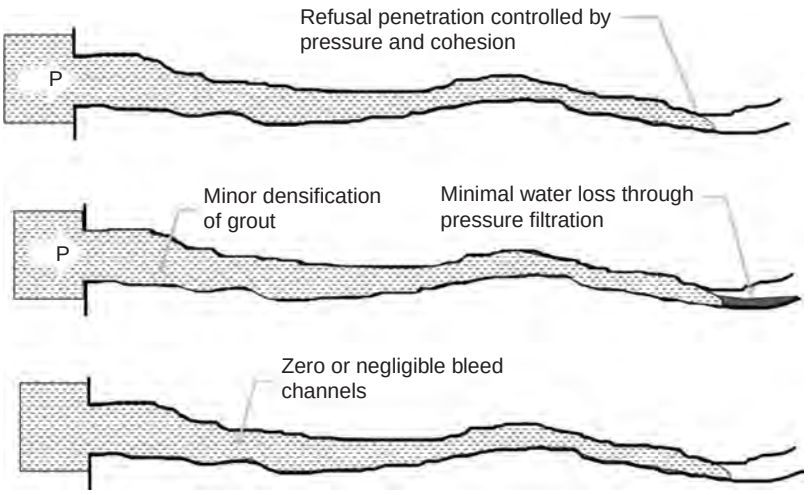


Figure 18.13 Performance of balanced stable grouts during and after injection.

It is common practise to use additives to improve the properties of the grout. Table 18.2 summarizes the additives and their beneficial and detrimental effects.

The main benefits of the additives are to reduce bleed or pressure filtration of water from the grout and to reduce the viscosity which increases the ability of the grout to penetrate the open joints and other defects in the rock. Grouts which bleed or lose water by pressure filtration are known as unstable grouts.

Figure 18.12 shows the effects of using unstable grouts. By comparison Figure 18.13 shows the performance of balanced stable grouts.

Table 18.2 Summary of additives, properties impacted and typical dosages (Chaqui, 2006).

Additive	*Description*	*Affects*	*Typical Dosage*
Super-Plasticizer	naphthalene sulphonate, ligno sulphonate, and melamine-based materials	Reduce the viscosity of the grout by inhibiting particle agglomeration. Enhanced strength and durability (Gause and Bruce, 1997). Typically retard the initial set of the grout.	0.5% to 2%
Bentonite[1]	sodium montmorillonite	Stabilizes the grout, increases its resistance to pressure filtration and increases its viscosity. Will reduce the ultimate strength (Deere, 1982) and (Littlejohn, 1982).	Less than 5%
Fly ash[2]	Type C and Type F Fly ash are pozzolanic materials.	Enhance resistance against pressure filtration and increase the durability of the cured grout.	Variable
Silica Fume (Micro Silica)	microfine powder (<1 micron)	Enhances resistance against pressure filtration, and increases durability and strength of cured grout by reducing its matrix porosity.	Less than 10% cement replacement
Thixotropic Agents/Gums[3]	Kelco-Crete/Welan Gum – high molecular weight biopolymers	Significantly enhance resistance to pressure filtration and make grouts thixotropic.	0.1% to 0.2%
Anti-Washout Agents[4]	Master Builder's Rheomac UW450	Enhances resistance to washout, reduces the pressure filtration coefficient, and makes the grout thixotropic.	0.2% to 1.0%
Hydration Controls[5]	Accelerators, Retarders and Hydration Inhibitors	Allow control of hydration process and manipulation of onset of initial set.	Variable

Notes.

[1] Bentonite – There is a wide variety of grades and types of bentonite. However, for most grouting operations, pure, chemically unaltered Wyoming sodium montmorillonite is optimal. The sequence and quality of mixing is critical (Jefferis, 1982). Bentonite should be hydrated for 12 hours prior to being used unless tests show that equivalent hydration can be achieved with a high shear mixer. Grouts with bentonite that has not been hydrated can be subject to durability problems due to cracking.

[2] Fly ash – It is important to note that Type C Fly ash expands and when used in dosages over 20% can cause durability problems in grouts.

[3] Thixotropic materials are characterized by viscosity which increases and decreases virtually instantaneously in response to the removal and application of shear. This property results in fluids which readily flow but are capable of suspending or stabilizing components.

[4] Anti-Washout Agents – This additive is not compatible with naphthalene sulphonate plasticizer and moderately compatible with Whelan gum and bentonite due to the sharp increase in the HMG viscosity. Typical proportioning is 0.2% to 1.0% by weight of cement.

[5] Hydration Controls – There are three distinct concepts:

a) Accelerators: There are several different kinds of accelerators the most common of which are sodium silicate and calcium chloride.

b) Retarders: Extend the gel and set times of grouts in a controllable fashion. Set times of several days are achievable with some of the more recent products.

c) Hydration Inhibitors: These are two-component systems involving the use of a stabilizer and an activator. The stabilizer forms a protective coating around the cement particles that stops the hydration process. When the activator is introduced to the grout dissolution of the protective barrier occurs, allowing the commencement of hydration and so normal crystal growth (Gause and Bruce, 1997).

Table 18.3 Relationships of different mix proportions of cement and water in grouts.

Water:Cement by volume	*Water:Cement by weight*	*Cement:Water by weight*	
6:1	4:1	1:4	(0.25)
4:1	2.67:1	1:2.67	(0.37)
3:1	2:1	1:2	(0.5)
1.2:1	1.33:1	1:1.33	(0.75)
1.5:1	1:1	1:1	(1.00)
1:1	0.67:1	1:0.67	(1.50)

18.2.3.2 *Water cement ratio*

(a) Cement-water grouts without additives

Early grouting practise was to use cement-water mixes without additives. There was an understanding that high water cement ratio mixes had better penetration. Bleed was controlled by reducing water cement ratios.

Cement-water grout mixes are usually designated by water cement (WC) ratios, with mixes ranging from 6:1 WC (by volume) to 0.6:1. Most grouting uses mixes of 2:1 WC ratio or less, as it is well documented that higher WC ratios yield unstable mixes (i.e. the particles settle quickly) and the grout is of poor durability (see Houlsby, 1985; Deere, 1982; Deere and Lombardi, 1985 and Alemo *et al.*, 1991).

Use of volumetric WC ratios has been traditional in Australian and USA practice because a bag of cement was taken to be one cubic foot and as such was an easy measure in the field. The relationship between volumetric and weight based ratio is approximate because it depends on bulking of the cement. Table 18.3 gives approximate relationships given by Deere (1982).

The selection of water-cement ratio is related to the 'stability' of the grout mix. As outlined by Deere (1982) stability is measured by a sedimentation test in which a litre of grout is placed in a standard 1000 ml graduated cylinder. At the end of 2 hours the volume of clean liquid that has formed at the top of the cylinder due to sedimentation is noted. This volume, expressed as a percentage of the total volume, gives the percentage 'bleeding' or sedimentation. Grout with a high WC ratio is 'less stable' and bleeding or sedimentation is large. ASTM C940 describes the test.

Lombardi and Deere (1993) and Stare *et al.* (2013) define stable grouts as having less than 5% bleed at 2 hours.

Modern practise is to use the pressure filtration coefficient (API 13B-1) which is measured in an API filter press. This is described in more detail in Table 18.4. For a balanced stable grout Stare *et al.* (2013) indicate that the pressure filtration coefficient Kpf should be $< 0.05\ \text{min}^{-0.5}$.

It was recognised early in the 1980s that addition of small percentages of properly hydrated bentonite improves the stability. This was later followed by addition of other additives as described in Table 18.2. WC ratios which should be used for cement-water grouting without additives are discussed in some detail by Houlsby (1977, 1978 and in WRC, 1981); Deere (1982); Deere and Lombardi (1985); Bruce (1982) and Bozovic (1985). For such grouts the authors have favoured the approach of Houlsby (1977,

Table 18.4 Standard field quality control tests for grouts (Chuaqui, 2006; Chuaqui and Bruce 2003).

Equipment	*Test*	*Description*
Marsh Funnel	Apparent Viscosity	The Marsh time of the grout can be measured in accordance with the method described in API Recommended Practice 13B-1 with a Marsh funnel and a calibrated container. The test is performed by filling the Marsh cone to the bottom of the dump screen and then measuring the time for 0.26 gallons (1 liter) of grout to flow through the funnel.
Penetrometer or Shear Vane or Lombardi plate	Cohesion and Time to Initial/Final Gelation	Either a penetrometer or shear vane type test will be used to measure the amount of time required for the grout to reach initial gelation (cohesion of 100 Pa) and final gelation (cohesion of 1000 Pa). Cohesion can be measured with a Lombardi plate (Lombardi, 2003)
API Filter Press	Pressure Filtration Coefficient	The pressure coefficient can be measured with an API filter press. The test is performed by pouring a 0.42 quart (400 ml) grout sample into the top of the filter press. The sample is then pressurized to 0.7 MPa. The test is run until all the water is expelled from the sample. The value of the pressure filtration coefficient is then calculated with the following equation: $Kpf = (\text{volume of filtrate})/(\text{volume of sample}) \times (\text{time in minutes})^{0.5}$
250-ml Graduated Cylinder – Glass	Bleed	The bleed capacity of the grout can be measured in accordance with the method ASTM C940 with a 0.26 quart (250 ml) graduate cylinder. The test is performed by pouring grout into the cylinder to the 0.21 quart (200 ml) level. The sample is then left undisturbed for two hours before the amount of bleed water is measured.
Baroid Mud Balance	Specific Gravity	The specific gravity of a grout can be measured in accordance with the method described in API Recommended Practice 13B-1 with a Baroid Mud Balance. The Baroid Mud Balance is a calibrated scale that is used to measure the specific gravity. Micromotion flow/density meters and hydrometers are also used in practice.
Vicat Needle	Initial and Final Set Times	The initial and final set times can be determined with the Vicat needle testing apparatus. The Vicat needle is set at the surface of the grout sample and released. Initial set is reached when the needle only penetrates 1 inch (25 mm). Final set is reached when the needle does not penetrate the surface of the grout sample.

1978, 1985) who recommended use of WC ratio (by volume) of not more than 3:1 and indicates doubts on long term durability if grout with WC ratios greater than 5:1 are used. He recommended use of the thickest possible mix at all times and suggests the following:

Starting Mix:

2:1	most sites
3:1	for rock <5 Lugeons
1:1	for rock >30 Lugeons
0.8:1	for very high losses
4:1	for heavily fractured, dry rock
5:1	rock above water table where excess water is absorbed by the dry rock.

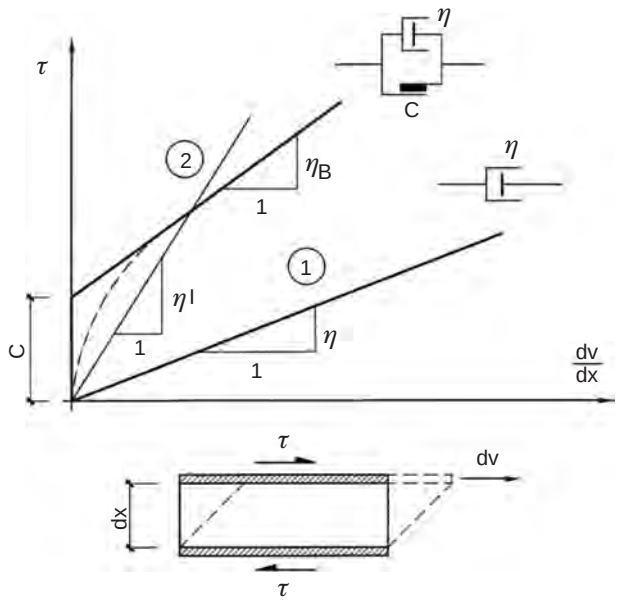


Figure 18.14 Rheology of Newtonian and Bingham fluids (Lombardi, 1985, 2003).

Thicken the mix:

- to deal with severe leaks
- after 1½ hours on the one mix with continued take (except for 1:1 and thicker mixes)
- if hole is taking grout fast, e.g. >500 litres in 15 minutes.

He indicates further that thickening should be in small increments, e.g. 3:1 to 2:1 and suggests reapplication of grout to the hole if the take has exceeded 0.25 litres/cm of hole for WC 2:1 or thinner or 0.5 litres/cm for WC 1:1 or thicker to fill voids of bleed water. He does not favour the use of bentonite to give stable mixes, preferring to use low WC ratios and bleeding of the water from the hole.

Grouts with higher WC ratio grouts will tend to settle to lower WC ratios. Hence higher viscosity results within the time of grouting and there appears to be little basis for using higher water contents to improve penetrability.

(b) Cement-water grouts with additives

Lombardi and Deere (2003) indicate that water: cement ratios of 0.67 to 0.8 by weight (about 1:1 to 1.2:1 by volume) are used for balanced stable grouts. Stare *et al.* (2013) indicate the range is 0.6 to 1.5 by weight.

18.2.3.3 *Rheological properties of grout*

As discussed in Lombardi (1985), from the rheological point of view water follows Newton's law and is called a Newtonian body and has viscosity only. A stable grout mix by comparison corresponds to a Bingham body and exhibits both cohesion (yield point stress), at which flow will begin, and viscosity. The slope of the line from the origin to any point on the Bingham curve is the apparent viscosity. These are shown in Figure 18.14. The cohesion and viscosity are dependent on water cement ratios, and the additives used in the grout.

If the fractures are sufficiently open to allow penetration of the grout, the distance to which the grout will penetrate is dependent on the fracture width, grout pressure and viscosity and the time taken in grouting. If grouting continues for sufficient time, the limit of penetration is determined by the cohesion or yield point stress. Lombardi (1985) showed that:

$$R_{max} = \frac{P_{max} a}{C} \tag{18.1}$$

where R_{max} = maximum radius of penetration (m); a = half width of the fracture (m); C = yield point stress (kPa); P_{max} = grouting pressure (kPa).

Lombardi (1985) presents a method for estimating the effect of grouting time. A feel for this can be obtained from the data in Deere and Lombardi (1985), which indicates that for medium to thick grouts approximately 75% of maximum penetration will occur in the first hour.

18.2.3.4 *High, medium and low mobility grouts*

Stare *et al.* (2013) describe three categories of grouts, each of which has different applications:

High mobility grouts are grouts used for conventional grouting of rock foundations. They are characterised by low cohesion and viscosity so they can readily penetrate open defects, and stability to control bleed or pressure filtration of the grout.

Medium mobility grouts are thick high mobility grouts with sand added to increase the apparent viscosity of the grout and limit penetration of grouts in karst foundations or very open defects in the rock.

Low mobility grouts are used for compaction grouting and are not relevant to this discussion.

Cement-water grouts are in principle high mobility grouts but are potentially unstable and suffer from bleed and pressure filtration particularly at high water-cement ratios. This is demonstrated in Figure 18.15.

Balanced stable grouts are cement – water grouts with additives which have an optimal blend of physical and rheological properties ("balanced") and are resistant to bleed and filtration ("stable").

18.2.3.5 *Field quality control testing of grouts*

Table 18.4 summarizes field quality control tests for grouts.

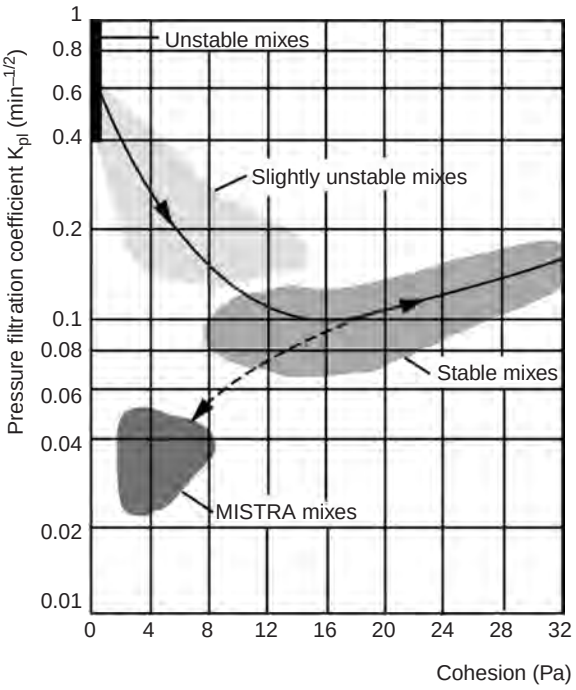


Figure 18.15 Relationships between resistance to pressure filtration and cohesion for different types of mixes. MISTRA refers to modified stabilized cement grout (Chacqui, 2006, from De Paoli *et al.*, 1992).

18.2.3.6 *Grout pressure*

There are several schools of thought on grout pressure:

(a) Those who limit grout pressures to below those which would lead to hydraulic fracture or jacking (The "conventional approach").
(b) Methods which seek to avoid hydraulic fracture but use higher pressures than the conventional approach, and use real time continuous monitoring of grout pressures and takes to detect hydraulic fracturing or jacking (the "enhanced conventional approach").
(c) Those who believe hydraulic fracture is preferred to promote the penetration of the grout.
(d) The Grout Intensity Number (GIN) method which uses a range of limiting pressures.

The authors are advocates of avoiding hydraulic fracturing of the rock, as we are concerned that the fractures opened by the grout will not all be filled by grout and the grouting may worsen the situation rather than improving it. We are not concerned that seepage still occurs through a dam foundation, provided the dam is designed to manage the seepage. The following discussion is based on avoiding hydraulic fracture.

The pressure, which can be applied, depends on the rock conditions (degree of fracturing, weathering, *in-situ* stresses and the depth of the water table) and whether grouting is carried out using a packer which is lowered down the hole at each stage (i.e. down-hole with packer grouting) or from the surface. The down-hole packer method allows progressively higher pressures.

The conventional approach

Houlsby (1977, 1978) and WRC (1981) present graphs to allow estimation of maximum pressures at the ground surface. These are based on the assumption that the maximum pressures at the base of the stage being grouted are given by:

$$P_B = \alpha d \tag{18.2}$$

where P_B = pressure at base of hole in kPa; α = factor depending on rock conditions; $\approx$70 for 'sound' rock; $\approx$50 for 'average' rock; $\approx$25 to 35 for 'weak' rock; d = depth of bottom of stage below ground surface in metres.

This allows for the weight of the overlying rock plus some spanning effect and has in the authors experience been found to be satisfactory.

The tendency for rock to fracture or 'jack' under grout pressures is reduced by using a relatively low pressure to start grouting and building up with time. Since much of the pressure in the grout is dissipated in overcoming the viscosity effects in the fracture, this limits the pressure transmitted to outer parts of the grout penetration. Houlsby in WRC (1981) suggests use of a starting pressure of 100 kPa (or less) for 5 minutes, then steadily increasing the pressure over the next 25 minutes until the maximum pressure is reached.

The occurrence of fracturing can be detected by sudden loss of grout pressures at the top of the hole, by increased take, surface leakage or by monitoring levels of the surface above the rock being grouted.

It is recommended by Houlsby that grouting is to 'refusal' and that the pressure is maintained for 15 minutes after this to allow time for initial set. Others suggest grouting until take is less than a certain volume in a time period; e.g. Stare *et al.* (2013) suggest 0.1 gpm (0.5 litres/minute) over a period of 5 minutes.

The enhanced conventional approach

As described in Stare *et al.* (2013) "rule of thumb" pressures may be appropriate for grouting the first two stages below the embankment and where the initial permeability is very high to reduce grout travel distance. They refer to USA practice where the rule of thumb limited pressure to about the overburden pressure. They advocate the use of higher pressures in other circumstances provided real time computer monitoring is used to detect the onset of hydraulic fracture and jacking.

The authors' support this view but emphasise that in their experience higher pressures than those described for the conventional approach should not be used unless there is real time monitoring of grout pressures and takes. It is not sufficient to rely on heave being detected and gross loss of grout without real time monitoring.

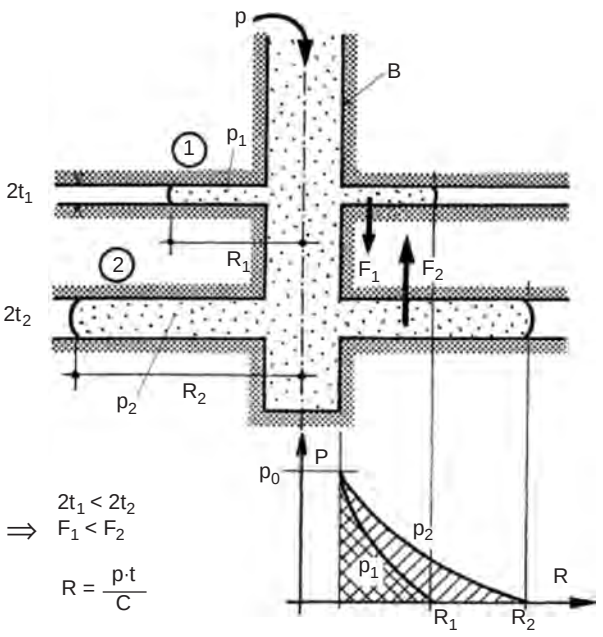


Figure 18.16 Pressure distribution and forces in a two joint system during grouting; B pressurised borehole. (1) Joint closes; (2) Joint opens; p = pressure; c = cohesion; 2t = opening; R = reach; F_1, F_2 = splitting forces (Lombardi, 2003).

Hydraulic fracture approach

The authors simply do not support this approach for the reasons given above. Grouting in dam foundations is about filling existing open fractures, not forcing grout into the ground by fracturing it.

The Grout Intensity Number (GIN) method

The GIN method is described in Lombardi and Deere (1993) and uses the principles outlined in Lombardi (1985) to allow the use of increased grout pressures while avoiding hydraulic fracture or heave. This recognizes that the pressures in an open defect reduce with distance from the grout hole and as a result the forces opening the defect are reduced. Figure 18.16 shows this schematically.

The GIN is defined as the product of the grout pressure (p) and the grout volume (V) taken in the hole, that is:

$$GIN = p \cdot V$$

It is usually expressed in units of bar.litres/metre of grout hole where a bar is 100 kPa. It is evaluated at a nil flow rate that is when the pumps have stopped.

Figure 18.17 shows Lombardi and Deere (1993) proposed limiting envelopes. They recommend using a moderate starting GIN value of 1500 bar.litres/metre but say trials are necessary on a dam site to select the appropriate GIN value.

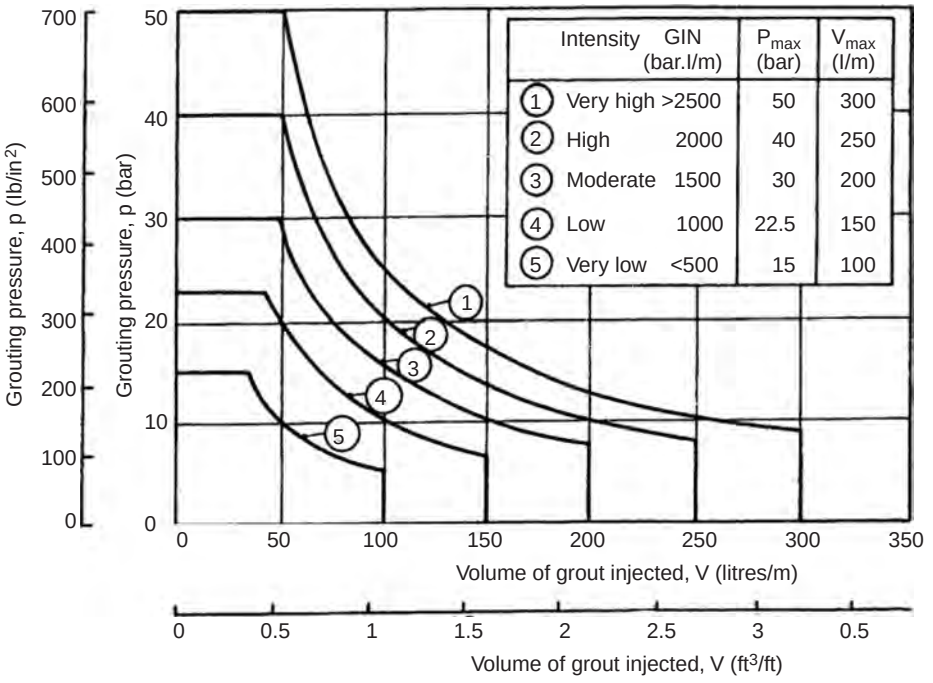

Grouting pressure, p (lb/in²)
Grouting pressure, p (bar)
Volume of grout injected, V (litres/m)
Volume of grout injected, V (ft³/ft)
Intensity GIN (bar.l/m) Pmax (bar) Vmax (l/m)
1 Very high >2500 50 300
2 High 2000 40 250
3 Moderate 1500 30 200
4 Low 1000 22.5 150
5 Very low <500 15 100

Figure 18.17 Set of "standard" GIN limiting curves, or guidelines (Lombardi, 2003, Lombardi and Deere, 1993).

Lombardi (2003) summarizes the principles of the GIN method as:

1. Define exactly the scope of the grouting works.
2. Do design, not specify, the grouting process.
3. Find out the "best mix" for the project by laboratory tests both from a technical and an economical point of view. Only stable mixes, generally with super-plasticiser, should be used.
4. Use only a single mix, "the best possible one", for all grouting stages in order to ensure the quality of the results and to simplify the procedures.
5. Define the parameters of the GIN curve: p_{max}, v_{max} and GIN value = p·V, taking into consideration the geological and rock mechanics parameters as well as the scope of the works and the related economy of the project.
6. Confirm the studies by field tests and check the works carried out by additional test grouting.
7. No water pressure tests to be carried out during the grouting works.
8. The split-spacing method is used in the GIN-Method as a self-adaptive and self-regulating procedure.
9. The use of stage lengths, increasing with depth below ground, is a way of speeding up the grouting works and of making some savings.

10. Inject water in dry, water-absorbing rock formations shortly before grouting, as a mean of avoiding a sudden blockage in the grouting process due to water losses from the mix.
11. The necessity of a new borehole is decided on the base of grout taken by the nearby ones.
12. Computer controlled procedures are a pre-requisite for optimal grouting."

Lombardi (2003) claims the main advantages of the method are:

- "Simplify the procedure in using a unique slurry and eliminating or at least reducing the waste of unused mixes;
- Reduce the risk of excessive hydro-jacking in eliminating the combination of high take at high pressure;
- Equalise approximately the reach of the grout at any grouting stage regardless of the rock mass quality, thus make the process more predictable;
- Produce a set of coherent data, which allows to judge the progress of the process as well as the result achieved and thus to optimise it;
- But, first of all the GIN-procedure is a self-adaptive one, which compensates for a substantial part the scatter of the natural conditions met in the rock mass."

The authors are satisfied with the principles of the method but are concerned that the lowest GIN curve in Figure 18.17 (curve 5) has a limiting pressure of 15 bars at low grout flows. This is 1500 kPa or equivalent to 30 m depth value for "average" rock in the Houlsby (1977, 1978) method. Such high pressures are almost certain to result in hydro fracturing in the first one or two stages of grouting (typically 0 to 12 metres) particularly in weathered or foliated rock or in horizontally bedded rock. The authors accept that the penetration of grout for low flows is small but have seen grouting at low flows and high pressures result in penetration of grout parallel to foliation where no open defect would have existed.

It is suggested therefore that trials are essential; that the lower GIN values be used until it is proven that higher values can safely be used; and that lower limiting pressures may be required than shown in Figure 18.17.

The authors do not agree with the use of "one mix" for a job. Maybe "one mix" simplifies the procedures, but we have found that most competent grouting specialists are very capable of being able to control the grouting when two or more specific mixes have been defined. There are clearly situations, such as very different geological conditions along the foundations of one dam or between the foundations of a "main" dam and a "saddle" dam, that warrant may very well warrant a change in the mix. The authors also do not agree with not carrying out water pressure testing as Lombardi (2003) suggests as we regard it as an essential means of proving the effectiveness of grouting.

If the GIN method is to be used it is essential that those involved read the papers which describe the method. Dr. Lombardi has on his web site copies of the papers listed above as well as a number of other papers discussing misunderstandings and criticism of the method. The authors believe that when the GIN method is used on a site and where there is careful and consistent supervision throughout the grouting operation, an excellent grouting result can be achieved. It is noticeable that on sites where the GIN method

Table 18.5 Guidelines for deciding on limits of effective grouting with Type HE (ASTM type III) Portland cement.

	No further grouting needed when			
Erodibility of foundation	*Water test value before grouting *(Lugeon)*[2]	*or <20% reduction in Lugeon value or grout take from previous stage*[2] *(Lugeon)*[2]	*or All grout takes (kg cement/m)*	*or Grout hole spacing (m)*
Low/non	<10	<20	<25	<1.5 m
High	<7	<15	<25	<1.5 m

[1] For rock with joints closer than 0.5 m.
[2] For Type GP (ASTM type I) Portland cement adopt Lugeon values 20% greater.

has been adopted that the site "housekeeping" is far better and that the site engineers and engineering geologists seem to be well "in control" of the whole operation.

18.2.3.7 *Recommended closure criteria for embankment and concrete dams*

From the review of criteria used by others, including Houlsby (1977, 1978, 1982a, 1985), WRC (1981), Deere (1982), Ewart (1985), Kjaernsli *et al.* (1992), the authors' experience and the discussion in Section 18.2.4 it is recommended that closure criteria should be based on:

- Lugeon value prior to grouting the hole.
- Grout take.
- The nature of the dam, its foundation and what is being stored in the dam.

These factors should be considered together to make decisions about whether further grouting is required.

It is not yet possible to quantify all these effects (and may never be), mainly because of the complexity of flow in fractured rock and the time dependent nature of cement grout properties. It is also not possible to make rigid rules – each case should be considered on its own merits. It is suggested that for grouting of the foundations of earth and earth and rockfill dams with Type HE (ASTM Type III) Portland cement the guidelines given in Table 18.5 be adopted. 'Erodible' foundations would include extremely to highly weathered rock and rock with clay filled joints which might erode under seepage flows.

For grouting with Type GP (ASTM type I) Portland cement, Lugeon values should be increased by 20% to account for the coarser nature of the grout particles. For micro-fine cement, grouting at values half those quoted would be reasonable from a grout penetrability viewpoint, but may not be justified by the benefit gained.

The overriding philosophy in these recommendations is that it is not possible to stop seepage by grouting, only to reduce it, and cement grout can only significantly reduce seepage when it can penetrate the fractures, i.e. in relatively widely spaced

open joints in moderate to high Lugeon value rock. As a rule-of-thumb for hard rock foundations, cement grout will not penetrate defects much smaller than about 0.25 mm (without resorting to hydraulic fracture, which we do not advocate), but water will still happily seep along such small open defects. The objective of the grouting operation should be to locate and fill these large fractures and to thereby avoid high and concentrated seepage flows; in that way, the foundation would end up with as close as practicable to a rock mass with uniform permeability. Crude estimates based on the results of the grouting along the saddle dam at Thomson Dam showed that the net permeability of the rock mass was reduced by between one-half and one order of ten, still worth the exercise when one considers what the total seepage could have been without the grouting.

Different criteria have been given for 'erodible' foundations, with some misgivings. It is better to acknowledge that, if foundations are erodible, grouting will not prevent erosion, only reduce erosion potential. With or without grouting, filters should be provided for erodible foundations under the downstream part of the dam to allow seepage water to emerge in a controlled manner without erosion of the foundation. For concrete dams, the foundation should be taken further down to non erodible rock.

It should be noted that, if the closure criteria detailed in Table 18.5 are followed, defects which are open about 0.25 mm or less will not be grouted.

The hydraulic shear stress imposed on the sides of the defect can be calculated using Equation 8.2 in Section 8.3.4.2. For a defect open 0.25 mm the shear stress is about 1 Pa for a gradient of 1.

As can be seen from Table 8.12 most soils have critical shear stresses > 10 and only dispersive soils and silts have critical shear stresses as low as 1 Pa.

Rock, even completely weathered rock will have much higher critical shear stresses so unless the defects are in-filled with dispersive soil, the likelihood of erosion initiating in the defect after grouting to the standards of Table 18.5 are extremely low or negligible.

For concrete faced rockfill, gravity concrete and arch dams seepage gradients are higher and there is some argument for using lower Lugeon values. However, the overriding consideration is that the cement will not penetrate fine fractures, so at most, the values quoted in Table 18.5 should be reduced by about 30%.

The question of whether the water is 'precious' or has a high contaminants content is not really relevant, since grouting to more restrictive Lugeon values as advocated by Houlsby (1986) will not result in significant reduction in seepage. In these cases it may be necessary to collect the seepage water downstream of the dam, either in a catch dam or in seepage collector wells and pump it back into the storage, but the costs in doing so could make such a move debatable. Pumping back the measured 5 l/s seepage at one of Adelaide's (South Australia) small, old puddle core dams was considered worthwhile; it has been done for many years.

A minimum grout hole spacing of 1.5 m is recommended because, if closer spacing is required to achieve a Lugeon value standard, the width of the resulting grout curtain is too narrow to significantly reduce seepage. Generally it would be better to control pore pressures by drainage rather than grouting. Clearly there may be exceptions where large takes are encountered in these holes, but as a general rule grouting with holes at closer than 1.5 m spacing is a waste of time and money.

It may also be a pointless exercise, given the potential for percussion-drilled holes to deviate from their planned position. At Thomson dam (Victoria), one 90 m deep hole was shown to be 27 m off line, which does pose the question as to the value of a deep single line curtain of closely-spaced holes.

The authors are of the opinion that it may be justified to reduce hole spacing below 1.5 m in some deeply weathered and closely fractured saddle dam foundations, where erodibility and control of seepage pore pressures may be critical for slope stability.

It should be noted that adoption of larger Lugeon value closure criteria at depth and/or higher grout pressures at depth may lead to secondary and tertiary holes not penetrating to the same depth as the primary holes.

A more detailed discussion on the selection of closure criteria is given in Fell *et al.* (1992).

If the GIN method is used the same Lugeon based criteria would apply with grout takes controlled by the GIN procedure.

18.2.4 Effect of cement particle size, viscosity, fracture spacing and Lugeon value on the effectiveness of grouting

Fell *et al.* (1992) presented a detailed discussion of the effect of cement particle size, viscosity, fracture spacing and opening and Lugeon value on the effectiveness of grouting. The main points are:

(a) *Cement grout particle size*. Cement grout is a suspension of the cement particles in water. The particles are mostly silt sized, but Portland cement will have a small percentage of fine sand particles. The grout particles aggregate in water to give a coarser distribution than the cement powder. The particle size distribution is affected by the addition of plasticizers, which act as deflocculating agents. This mainly affects the finer particles. With plasticizers, Type A and C Portland cements (equivalent to AS 3972 GP and HE; ASTM C 150 type I and III) have a maximum size of about 0.08 mm and 0.05 mm respectively, while the microfine cements tested had a maximum size of about 0.02–0.025 mm. These are consistent with other published data.

Minimum fracture opening which will accept grout. There is a general consensus, (e.g. Mitchell, 1970; Karol, 1985) backed up by some tests carried out by Tjandrajana (1989) under the first author's supervision, that the minimum opening fracture into which grout will continue to penetrate is about $3D_{100}$ where D_{100} is the sieve size for which all the grout particles are finer. Houlsby (1982b, 1986) stated that grouting was difficult with a fracture width less than 0.5 mm, and gave a lower limit of 0.2 mm. Littlejohn, (1982, 1985) suggested a lower limit of 0.16 mm or narrower than $5D_{100}$. From this and the data on particle size in (a) it may be concluded that the minimum defect opening into which grout will penetrate is between 0.16 mm and 0.25 mm. Figure 18.18 shows approximate relationships between the defect opening and Lugeon value determined using the method of Louis (1969). The details are given in Fell *et al.* (1992).

Based on these plots and the limiting opening into which grout can penetrate, an approximate estimate of the minimum Lugeon value rock which can be grouted is as shown in Table 18.6. This indicates that rock masses giving

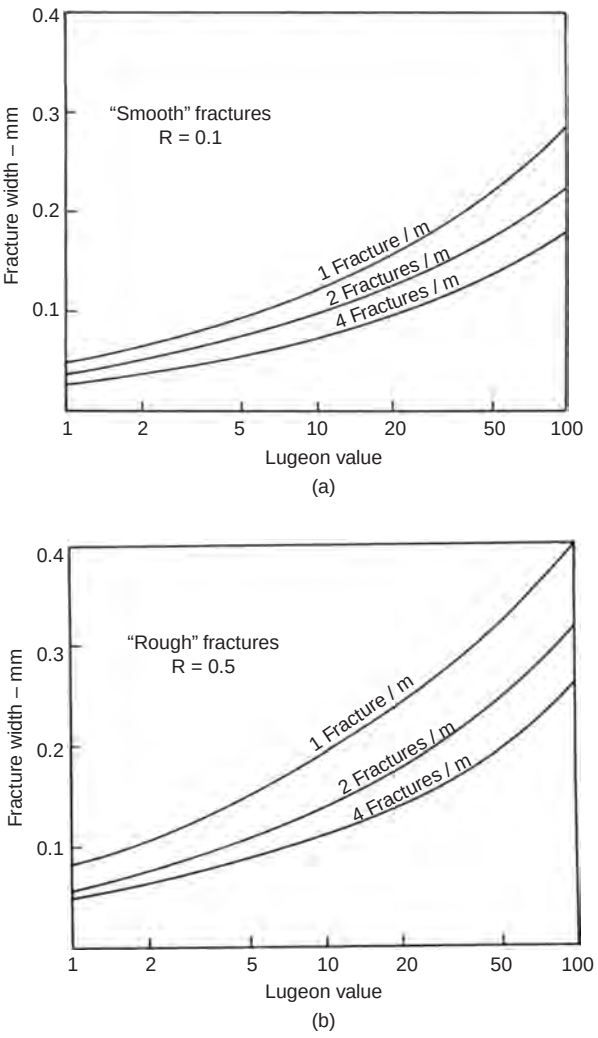


Figure 18.18 Defect spacing and opening versus Lugeon value based on Louis (1969). (a) Smooth defects, (b) Rough defects.

some quite high Lugeon values cannot be grouted, which may seem contrary to experience. However, the larger values (15 to 30 Lugeons) occur when there are two or more open defects (e.g. joints) per metre. This condition may not be met often, even when there are three or more defects per metre, as many defects may not be open.

(b) *Distance grout will penetrate*. The distance grout will penetrate into open defects in the rock depends on whether the defect is sufficiently open to enter the defect, the persistence of the defect; the opening of the defect and whether the opening is uniform; the grout rheology; grout pressure and the grouting time.

Table 18.6 Estimated minimum Lugeon values indicative of rock which will accept cement grout (based on Fell *et al.*, 1992).

	Minimum Lugeon value which can be grouted		
Cement	*1 fracture/m*	*2 fractures/m*	*4 fractures/m*
Type A (AS 3972 type GP; ASTM type I)	8	16	32
Type C (AS 3972 type HE, ASTM type III)	5	10	20
MC-500 (microfine)	3	5	10
Type A with plasticizer	8	16	32
Type C with plasticizer	5	10	20
MC-500 (microfine) with plasticizer	1	2	4

Notes: 1. Fractures are assumed 'rough'; 2. Fractures assumed to be the same width; 3. One Lugeon is a flow of 1 litre/minute/metre of borehole under a pressure of 1000 kPa. In a 75 mm diameter borehole it is approx. 1.3×10^{-7} m/sec equivalent permeability; 4. Grout is assumed to have been treated with plasticizer.

Table 18.7a Theoretical radius of penetration (metres) as a function of defect opening, grout properties, time, for a grout pressure of 2000 kPa (based on Deere and Lombardi, 1985).

Grouting time	*Thin grout*	*Medium grout*	*Thick grout*
Defect opening 0.5 mm			
15 minutes	28 m	17 m	13 m
1 hr	49 m	26 m	20 m
4 hr	81 m	34 m	24 m
8 hr	100 m	36 m	25 m
Defect opening 1 mm			
15 minutes	43 m	32 m	25 m
1 hr	80 m	50 m	38 m
4 hr	100+ m	67 m	48 m
8 hr		72 m	50 m

Table 18.7a summarizes the theoretical radius of penetration of grout for a grout pressure of 2000 kPa for defects open 0.5 mm and 1.0 mm. This data is taken from Deere and Lombardi (1985). As 2000 kPa is a high pressure for grouting at shallow depths the figures in Table 18.7b have been calculated for a grout pressure of 300 kPa assuming that the radius of penetration is proportional to the grouting pressure (as indicated in Equation 18.1). Grout pressure of 300 kPa is the upper bound of recommended pressures in "average ground" according to the Houlsby (1977, 1978) approach at a grout depth of 6 m.

It should be noted that 0.5 mm open defects at 1 m spacing would result in lugeon values >100 based on Figure 18.18.

These are very large grout penetrations compared to what is seen in practice. This is due to:

(a) The simplifying assumptions of continuous open defects of constant opening. As pointed out by Lombardi (2003) real defects are more likely to be irregular

Table 18.7b Theoretical radius of penetration (metres) as a function of defect opening, grout properties, time, for a grout pressure of 300 kPa (based on Deere and Lombardi, 1985).

Grouting time	*Thin grout*	*Medium grout*	*Thick grout*
Defect opening 0.5 mm			
15 minutes	4 m	3 m	2 m
1 hr	7 m	4 m	3 m
4 hr	12 m	5 m	4 m
8 hr	15 m	5 m	4 m
Defect opening 1 mm			
15 minutes	6 m	5 m	4 m
1 hr	12 m	7 m	6 m
4 hr	15+ m	10 m	7 m
8 hr		11 m	8 m

Assumptions, Thin grout – cohesion 2 Pa, viscosity 0.68×10^{-6} m·sec;
Medium grout – cohesion 7.5 Pa, viscosity 1.30×10^{-6} m·sec;
Thick grout – cohesion 10.5 Pa, viscosity 1.85×10^{-6} m·sec

Table 18.8 Reduction in permeability achieved by grouting (from data in Houlsby, 1985).

		Average permeability before grouting – Lugeons				
Dam	*Jointing and rock type*	*24 m*	*12 m*	*6 m*	*3 m*	*1.5 m*[1]
Copeton	Wide continuous in granite	21	10	9	8	6
Toonumbar	Medium, sandstone, conglomerate and shale		38	30	25	10
Glenlyon	Medium-close, claystone, calcite and quartz veins		48	42	37	15
Chaffey	Very close, jasper, Siltstone and chert		19	15	18	24

Note: 1. Grout hole spacing (m).

in shape so grout will accumulate at the narrow points as shown in narrower points and prevent greater penetration unless hydro fracturing is used to open the defects.

(b) Much grouting which has been carried out on existing dams has followed the Houlsby (1977, 1978) approach of limiting grout pressures to avoid hydraulic fracture or "jacking". Table 18.8 summarizes data for a number of large dams constructed in Australia in the 1970s using these principles. It can be seen from this data that at Copeton Dam grout penetration into continuous open joints was greater than 12 m; at Toonumbar and Glenlyon, little reduction in permeability was achieved until the holes were at 3 m spacing, indicating grout penetration of 1.5 m to 2 m in the medium-close jointed rock; and at Chaffey, no benefit was achieved even with the holes at 3 m spacing indicating grout penetrations was less than 1.5 m in the very closely jointed rock.

Table 18.9 Approximate likely maximum penetration from the borehole of cement grout in open defects (metres) assuming conventional limiting grout pressures.

	Spacing of open fractures		
Lugeon value	*1 m*	*0.5 m*	*0.25 m*
100	20 m	12 m	4 m
50	12 m	3 m	2 m
20	3 m	1.5 m	1 m
10	2 m	1 m or NP	NP
5	1 m or NP	NP	NP
1	NP	NP	NP

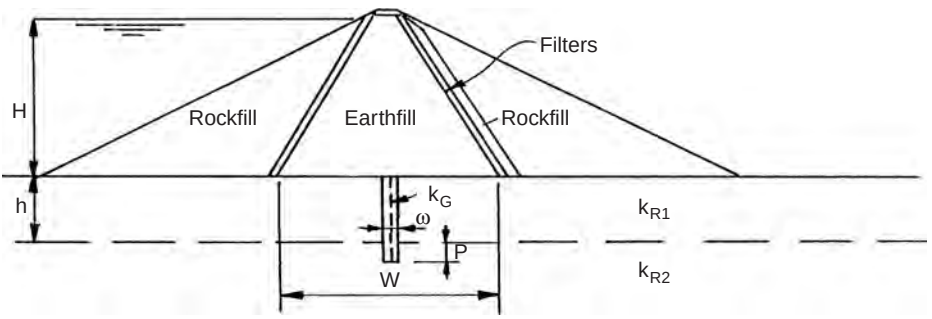


Figure 18.19 Generalised model for seepage through a dam foundation.

Based on this and the authors experience on a number of grouting projects, it is considered reasonable to conclude that the penetration of AS 3972 type GP (ASTM type I) and AS 3972 type HE (ASTM type III) cement grouts will typically be of the order of those shown in Table 18.9. In the table NP indicates the grout will not penetrate the fractures because the defects are not sufficiently open.

If larger pressures are used allowing hydraulic fracture or "jacking" grout will penetrate further, but may not result in further reduction in permeability as the fine defects within the rock are unlikely to be all filled with grout.

18.2.5 The effectiveness of a grout curtain in reducing seepage

Figure 18.19 shows a simplified typical embankment and foundation section with a low permeability core, high permeability rockfill shoulders and a foundation consisting of an upper layer of permeable rock overlying 'impermeable' rock.

Figure 18.20 presents the results of an approximate seepage analysis to show the effectiveness of a grout curtain in reducing the amount of seepage beneath the embankment.

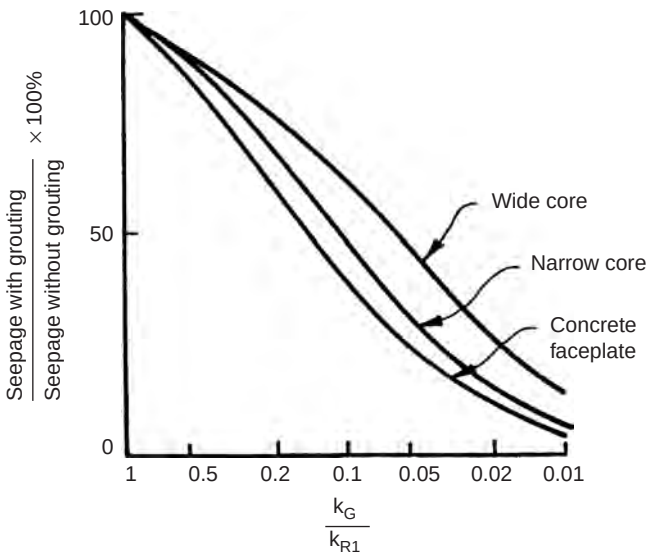


Figure 18.20 Effectiveness of grouting in reducing seepage in a dam foundation.

The analysis has been based on:

H = 60 m all cases
W = 60 m for wide earthfill core, 20 m for narrow earthfill core and 6 m for concrete face plinth = 6 m all cases
h = 20 m all cases

Unless the grout curtain has a permeability at least 10 times lower than the un-grouted rock the reduction in seepage is less than 50%. So for 20 Lugeon rock the grout curtain must be less than 2 Lugeons to achieve 50% reduction in seepage. As pointed out above, it is doubtful whether such a low permeability is achievable with cement grouting because the cement will not penetrate rock with such low Lugeon value.

For high permeability rock (say 100 Lugeon) with wide fracture spacing, the effective width of the curtain should be larger (say 20 m) and the permeability could be reduced to that of the secondary fracture system. If this were, say, 5 Lugeons, then the seepage would be reduced to about 20% of the un-grouted value. If the permeability of the secondary fractures was, say, 2 Lugeons, the seepage would be reduced to 5% to 10% of the un-grouted value.

18.2.6 The depth and lateral extent of grouting

So far as is practicable grout holes should be taken to the depth at which the Lugeon closure criteria are achieved. In near horizontally layered rock this might be clearly

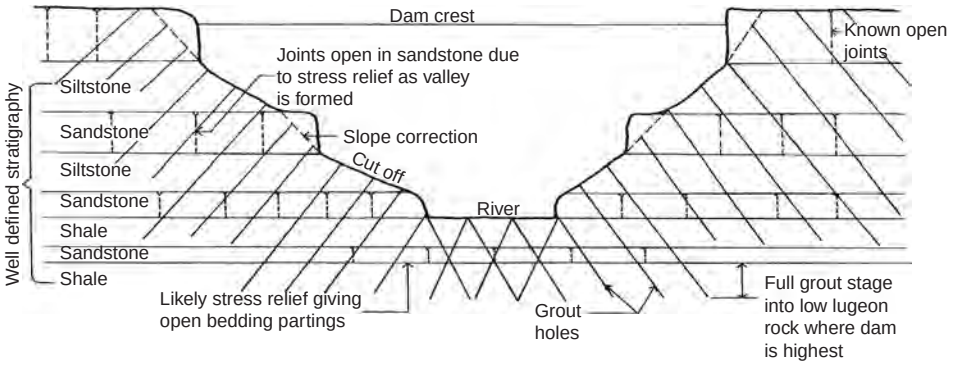


Figure 18.21 Example of grout curtain in a simple sedimentary rock environment.

identifiable as a lower permeability zone of rock beneath the valley floor, e.g. siltstone within interbedded siltstone and sandstone, but may be at different depths around the dam abutments due to the influence of stress relief, weathering and rock types. Figure 18.21 gives an example where the stratigraphy is simple and clearly known.

The depth of higher Lugeon value rock is seldom known with any accuracy and the grout holes must penetrate to at least one stage below the estimated base level to prove that the necessary conditions have been met.

The use of 'rules of thumb' to determine the depth of grouting is not recommended as there is no logical basis for them.

If such rules are applied they will either yield holes which are too shallow, particularly in the abutments, resulting in a grout curtain which only partially penetrates the permeable foundation, giving only minor (i.e. typically less than 10%) reduction in seepage due to increased seepage path length or, more commonly, holes which are much deeper than required to reach lower permeability rock, particularly in the river section of the foundation where the dam is highest.

For most dams, grouting will extend up the abutments to where full supply level intersects the base of the permeable zone. However, where the dam abuts a spillway, the grout curtain may be connected into the curtain under the spillway and/or if the dam abuts a relatively narrow permeable ridge, the grout curtain may be extended into the ridge (Figure 18.22a).

Extension of the grout curtain further into an abutment is only necessary if the stability of the ridge or abutment is in question and there is a need to control piezometric pressures, or if the abutment is highly permeable.

In such highly permeable rock, careful consideration must be given to the lateral extent of grouting, or seepage may bypass the end of the grout curtain. Figure 18.22(b) shows the grout curtain for a 20 m high water storage dam in coal measure rocks, which have been intruded by a highly permeable dolerite sill. The grout curtain was successfully constructed with excellent closure, but when the dam was filled the reservoir leaked significantly (water level dropped 25 mm/day) because water was entering and flowing along the dolerite sill, completely bypassing the ends of curtain. This groundwater movement was proven conclusively by piezometers.

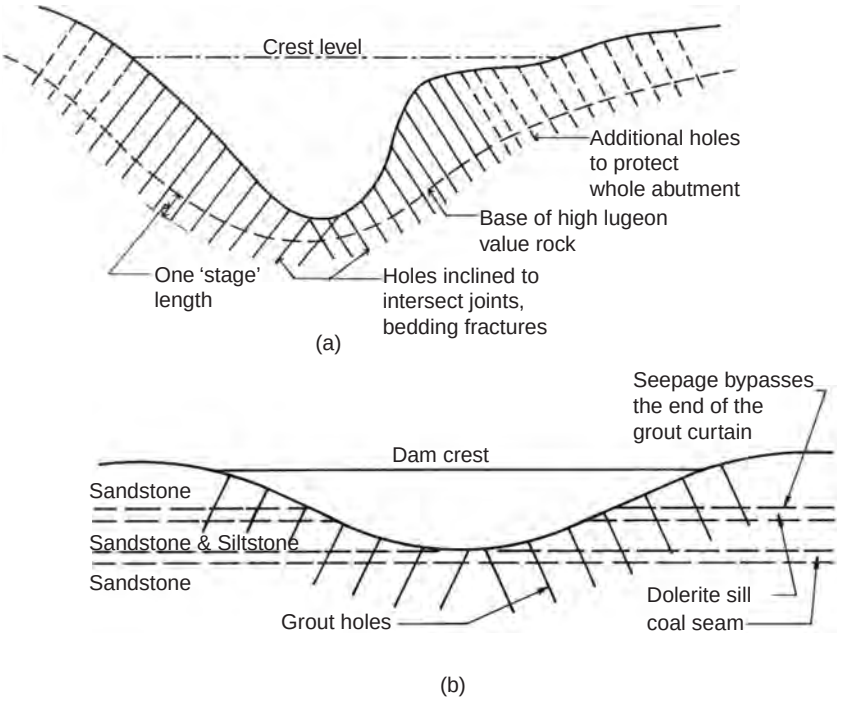


Figure 18.22 (a) Example of grout curtain extended beyond dam crest; (b) Example where seepage bypassed the end of the grout curtain.

18.2.7 Grout hole position and orientation

For earthfill and earth and rockfill dams the grout curtain will usually be located in the centre of the cutoff trench, which in turn will usually be located at the centre or upstream of centre of the earthfill zone. Houlsby (1977, 1978) presents an argument for positioning grout curtains upstream of centre of the earthfill zone, to maintain a seepage pressure head greater in the earth core than in the foundation. This concept has merit in principle but in practice flow nets within the earthfill and foundation are influenced by different permeability and positioning the curtain upstream of the centre of the core may not achieve the desired objective. In any case, experience based on pressure measurements across several cores of embankments dams would suggest that the pressure drop is often very close to a straight line distribution from the full reservoir head to the tailwater level or zero on the downstream side.

For concrete face rockfill dams the grout curtain will be positioned in the plinth, usually, but not always, with a line of consolidation grout holes upstream and downstream of the curtain to give better grouting protection close to the surface near the plinth, where gradients are high.

For concrete gravity dams (including spillways) the grout curtain should be positioned close to the upstream face so that the drainage holes can be positioned downstream of the grout curtain but still close to the upstream face to reduce uplift

pressures as much as possible. It should be noted that for these structures the drain holes are critical in reducing uplift pressures and are more important than grouting. Grouting alone cannot be relied upon to reduce uplift pressures (see the above comments on cores of embankment dams). Note that a popular way of handling grouting for an RCC dam is to do that work from a small plinth along the heel of the dam to avoid delaying the RCC placement operation. The authors do not favour such an approach, but if it is to be done that way, then the plinth should be thick enough to avoid a crack forming against or near to the upstream face of the dam.

Grout holes should always be oriented to intersect the major fracture sets, particularly those which are near parallel to the river. In most cases this will mean that the holes are inclined from the vertical as shown in Figures 18.21 and 18.22. They may also need to be inclined upstream to intersect joints perpendicular to the river.

Far too often the authors have to intervene in projects for which they are reviewers where those involved have specified vertical holes where the major defects to be grouted are vertical. Using vertical holes in such situations e.g. as in Figure 18.21 is virtually useless.

18.3 SOME PRACTICAL ASPECTS OF GROUTING WITH CEMENT

18.3.1 Grout holes

It has been accepted practice for many years in Australia and other countries to use wet percussion drilling for grout holes. Holes are a 30 mm minimum diameter, usually 50 mm diameter and seldom larger than 60 mm.

Percussion drilling is the quickest and cheapest type of drilling and is satisfactory except where the nature of the rock is such as to create a sludge or stiff clayey 'plug' which blocks the fractures in the rock. In such rock rotary drilling using roller bits or plug diamond bits may be used. It is sometimes argued that rotary or diamond drilling will be less likely to block the fractures. If holes are washed carefully after drilling it is doubted whether the drilling method really has much effect. This was proven in the quartzite and phyllite foundations of Blowering dam. Sometimes the geology of the foundations can become a problem for some drilling methods. The earlier mentioned 27 m off line incident at Thomson Dam appears to have been caused by the drill bit going through a softer bed of rock (interbedded sandstone and siltstone) to hit a harder bed, when the drill started to work itself towards being normal to that hard bed.

Stare *et al.* (2013) discuss drilling methods. They suggest that top-hole percussion with water flush is the preferred method for holes up to 50 m deep. However they point out that holes may deviate from the required orientation, and indicate that this can be addressed to some extent by using guide tubes at the top of the hole.

Stare *et al.* (2013) indicate that bottom – hole percussion methods with water flush are satisfactory for deeper holes.

Air flush percussion methods should be prohibited because they promote blockage of the open defects with the cuttings and may fracture the rock by the high air pressures.

Washing of the grout hole before grouting is essential to remove cuttings which have clogged the fractures. This is done by lowering a specially constructed washout bit which directs water under pressure against the sides of the hole. Figure 18.23 shows an example of such a bit, which may also be used to flush grout from a hole after grouting.

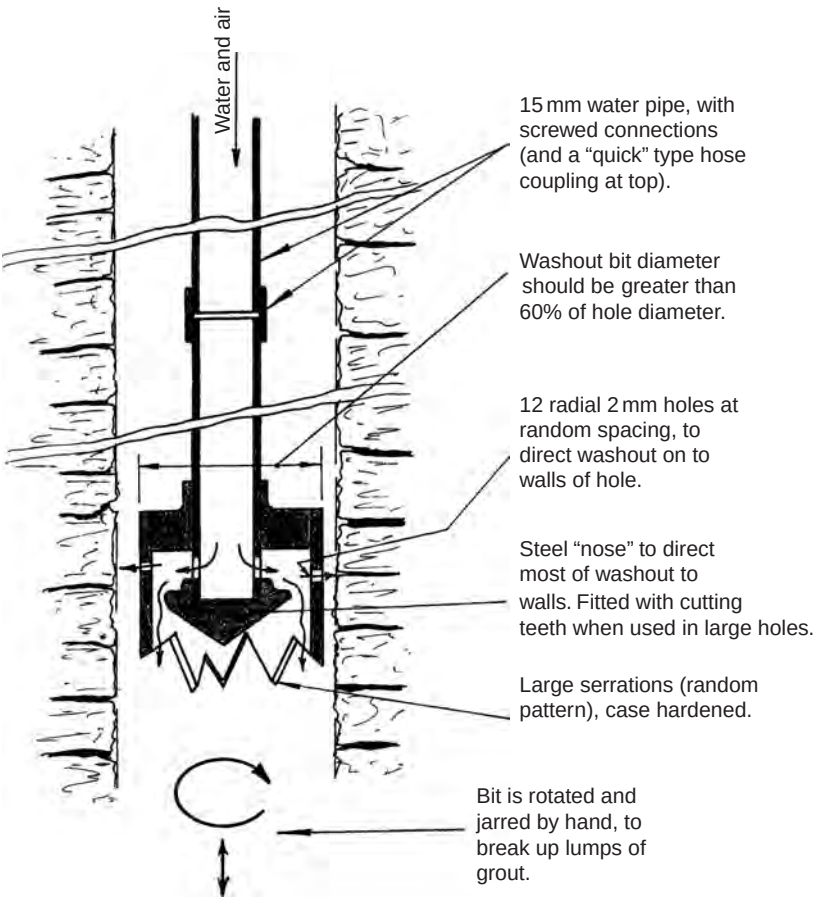

Water and air
15 mm water pipe, with screwed connections (and a "quick" type hose coupling at top).
Washout bit diameter should be greater than 60% of hole diameter.
12 radial 2 mm holes at random spacing, to direct washout on to walls of hole.
Steel "nose" to direct most of washout to walls. Fitted with cutting teeth when used in large holes.
Large serrations (random pattern), case hardened.
Bit is rotated and jarred by hand, to break up lumps of grout.

Figure 18.23 Grout hole washing bit (WRC, 1981).

Weaver (1993) advocates stopping drilling immediately water loss occurs because if the hole is drilled further, cuttings from the drilling may block the feature into which the drill water was lost. After grouting, drilling to lower stages is carried out.

18.3.2 Standpipes

Standpipes are used for most grouting operations where packers are not used. The standpipe consists of a threaded galvanized pipe just larger than the drill size, set approx. 0.6 m into the rock as shown in Figure 18.24.

Standpipes will always be required where the rock near the surface is fractured or weak, precluding use of a packer. Standpipes have the advantages of preventing clogging of the hole by debris from adjacent areas and providing easy hook-up for grouting. Note that the part of the standpipe projecting from the surface has to be removed before placing earthfill.

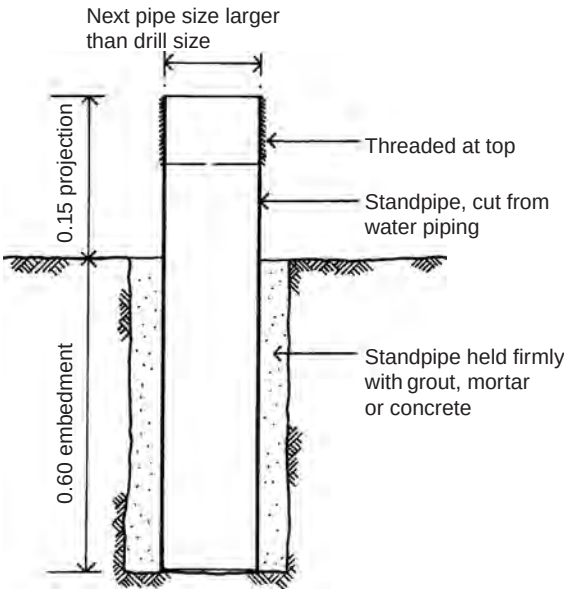


Figure 18.24 Grout standpipe.

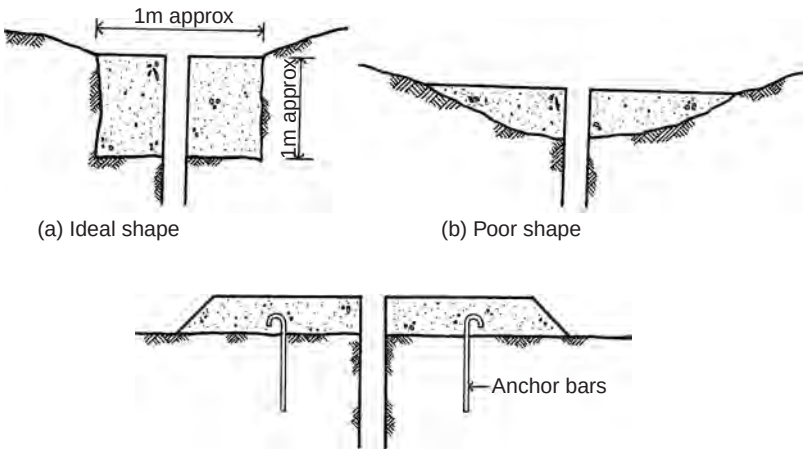


Figure 18.25 Grout caps (a) Ideal shape, (b) Poor shape, (c) Grout cap above cutoff level.

18.3.3 Grout caps

Concrete grout caps (Figure 18.25) are required when grouting closely fractured or low strength rock in the situation where a standpipe cannot be sealed into the foundation and/or leakage of grout to the surface will be excessive. The grout cap also provides a

better cutoff through the upper part of the foundation than is practicable with grouting from standpipes.

Grout caps should be excavated into the foundation rock without explosives and if practicable should be of square or rectangular section as shown in Figure 18.25(a). This gives good resistance to lifting of the cap under the pressure of grouting.

The wider/shallow shapes shown in Figure 18.25(b) and (c) are prone to displacement during grouting and may need to be anchored into the foundation with steel dowel bars grouted say 2 m into the rock.

Some organisations, e.g. Rural Water Commission of Victoria, have favoured construction of a grout cap in all foundations, constructed above the cutoff level as shown in Figure 18.25(c). This arrangement requires anchor dowels in all cases. The arrangement is justified on the basis of allowing grouting of all the rock, i.e. right to the surface, and increasing the resistance to erosion along the cutoff surface by providing a more tortuous path for water, but has the disadvantage that it interferes with compaction of earthfill in the cutoff trench. Such grout caps are not recommended by the authors.

In closely fractured rock there is often some advantage in applying a layer of slush concrete or shotcrete to the cutoff surface before grouting. This allows use of slightly higher grout pressures in some rock types, prevents excessive leakage to the surface and generally facilitates grouting. It also prevents damage to the cutoff surface by construction equipment during the grouting operation. Increasing grout pressures even with a surface cover of concrete (shotcrete) should be done with care as lift-off of the cover can be initiated unwittingly.

18.3.4 Grout mixers, agitator pumps and other equipment

Houlsby (1977, 1978 and in WRC, 1981) strongly advocates the use of high speed, high shear, 'colloidal' mixers. This opinion is supported by US Corps of Engineers (1984), Deere (1982), Gourlay and Carson (1982), Bruce (1982) and Stare *et al.* (2013). Figure 18.26 taken from WRC (1981) shows the principle of some suitable mixers. The most commonly used mixers operate at 2000 rpm and 1500 rpm. The very high speed facilitates mixing by separating cement particles from each other and wetting the surface of each particle. It is claimed (WRC, 1981) that mixing with such high-speed mixers "produces a grout which more closely resembles a colloidal solution rather than a mechanical suspension." While relatively speaking the grout may settle more slowly, one should not lose sight of the fact that it still is a suspension of cement particles which do settle with time although additives reduce this.

Having mixed the grout it is usually transferred to an agitator from which it is pumped to the grout holes. The agitator is relatively slow speed and designed to prevent the cement particles from settling. Figure 18.27 (from WRC, 1981) shows the principle of the desired arrangement.

The quantity of grout injected is measured at the agitator.

Grout is pumped from the agitator using grout pumps. Houlsby (1977, 1978) and Deere (1982) indicate that in Australian and USA practice it is common to use 'mono' (or 'moyno') helical screw pumps because they provide a constant pressure, are rugged and readily maintained. A grout bypass valve at the hole is required as shown in Figure 18.28.

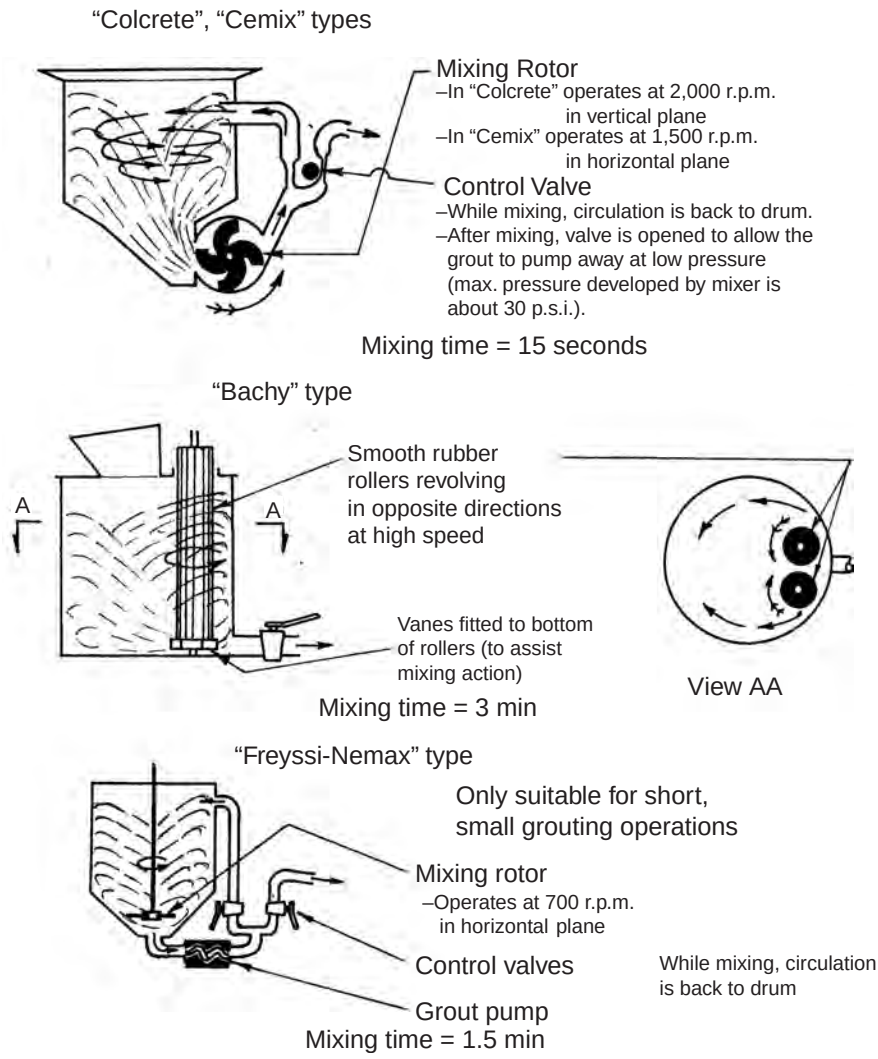

"Colcrete", "Cemix" types
Mixing Rotor
–In "Colcrete" operates at 2,000 r.p.m. in vertical plane
–In "Cemix" operates at 1,500 r.p.m. in horizontal plane
Control Valve
–While mixing, circulation is back to drum.
–After mixing, valve is opened to allow the grout to pump away at low pressure (max. pressure developed by mixer is about 30 p.s.i.).
Mixing time = 15 seconds
"Bachy" type
A
A
Smooth rubber rollers revolving in opposite directions at high speed
Vanes fitted to bottom of rollers (to assist mixing action)
View AA
Mixing time = 3 min
"Freyssi-Nemax" type
Only suitable for short, small grouting operations
Mixing rotor
–Operates at 700 r.p.m. in horizontal plane
Control valves
Grout pump
While mixing, circulation is back to drum
Mixing time = 1.5 min

Figure 18.26 Principles of operation of grout mixers (Houlsby, 1977, 1978 and WRC, 1981).

Gourlay and Carson (1982) and Bruce (1982) point out that in British and European practice there is a preference for ram type pumps, as it is believed the pulsating pressure assists in preventing clogging of the fracture opening by coarser grout particles. Deere (1982) also indicates a preference for this type of arrangement, largely because it obviates the need for a bleeder valve at the hole and/or recirculation line. As pointed out by Gourlay and Carson, there is a lack of hard evidence to support either preference, and one should be willing to use the equipment readily available. Houlsby (1977, 1978) advocates the use of a recirculation line of the smallest diameter practicable to keep grout velocities high and avoid blockage. He suggests the maximum size

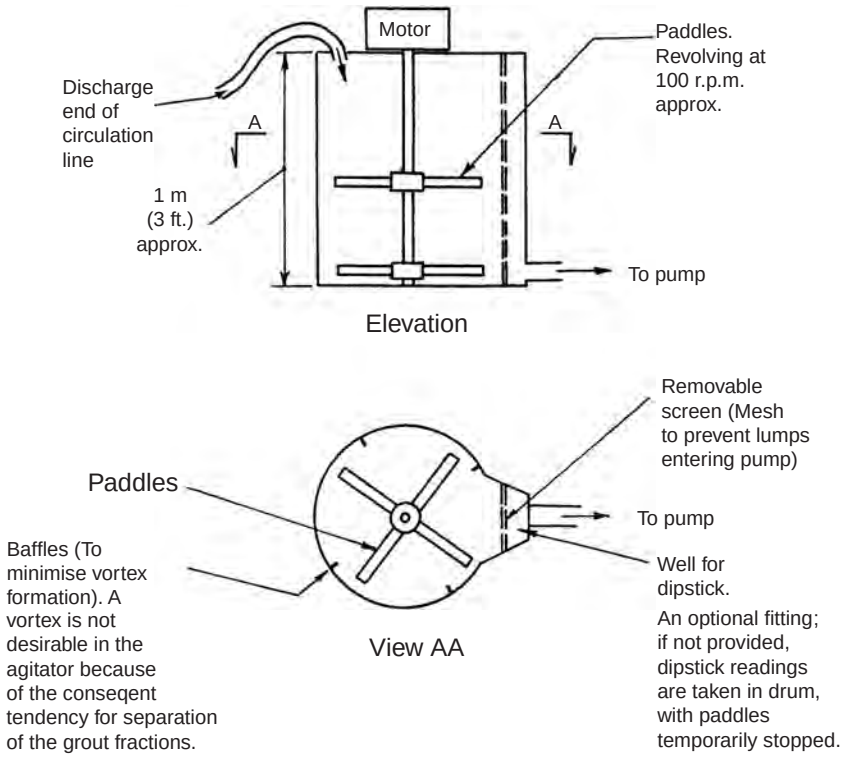


Figure 18.27 Principle of grout agitators (Houlsby 1977, 1978 and WRC 1981).

is 25 mm diameter. Deere (1982) suggests that the recirculation line is used in US practice because of the then traditional use of high water/cement ratio grouts which were 'unstable' (i.e. settled out) and the requirement for a bypass valve to control injection pressure.

Details of bleeder valves and bypass, flow meters, pressure gauges and standpipe fittings are given in Houlsby (1977, 1978), WRC (1981) and Gourlay and Carson (1982).

Packers for use in grout holes are either mechanically operated from the surface by a screw device, or inflatable, expanded against the side of the hole by compressed air or water. The length of 'seal' formed by mechanical packers is often only 0.3 m, compared to 0.5 m to 1.5 m for the inflatable type, so leakage past the packer is more likely with the former. Mechanical packers would usually only be adopted if rock conditions resulted in puncturing of inflatable packers.

18.3.5 Monitoring of grouting program

It is absolutely essential that detailed records of the grouting operation are kept. This is necessary to allow progressive development of the grouting operation, e.g. decision making on hole depths, whether closure holes are required or whether grout mixes

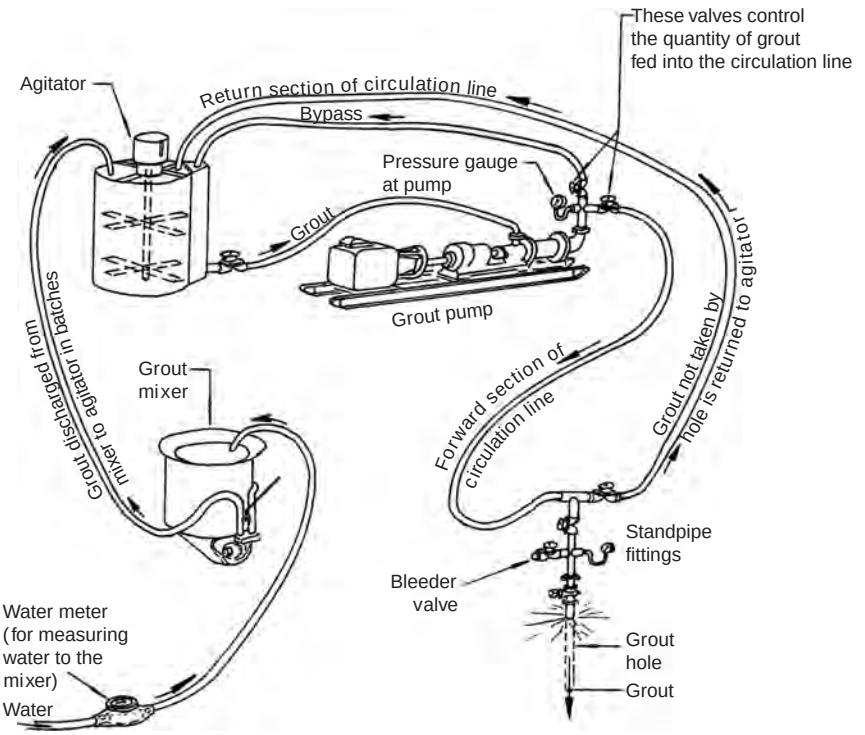


Figure 18.28 Typical arrangement of grouting equipment (Houlsby, 1977, 1978, and WRC, 1981).

should be changed. In most cases, detailed records will also be needed for payment purposes. Matters recorded should include:

- Hole locations, orientation, depths.
- Stage depths.
- Water pressure test value for each stage prior to grouting, including maximum pressure used.
- Grout mix(es).
- Grout pressures, takes at for example 15 minute intervals and then in summary.
- Grouting times.
- Leaks, uplift.
- Total grout takes for each stage.
- Amount of cement in these takes.
- Cement takes/unit length of hold.

This data will be collected largely by foremen but an engineering geologist should be involved continuously to interpret the progress of the grouting, relate this to the geotechnical model of the foundation and make decisions on closure etc. This will

invariably involve keeping records on sections along and across the grout centreline with geological data superimposed.

For grouting using enhanced traditional and GIN methods real-time computer monitoring of pressure, flow rate and take, with the ability to plot pressure vs time; flow rate vs time; take vs time; pressure vs take and for GIN estimated penetrability vs take. These are essential to monitor for hydraulic fracture and jacking. Note that because this system has real-time monitoring does not mean that careful supervision by the site team can be eased off. Such IS NOT the case, as was emphasised earlier in this chapter.

18.3.6 Water pressure testing

Before grouting each stage, a water pressure test should be carried out. This is done using a method similar to that outlined in Chapter 5, but using a simplified procedure, e.g. applying only one pressure (100 kPa) for 15 minutes, taking flow quantities at 5, 10 and 15 minutes to estimate the Lugeon value.

Stare *et al.* (2013) advocate the use of the apparent Lugeon value which is estimated from the grout takes with a correction for the viscosity by using the Marsh cone values. The authors do not see this as a substitute for water pressure testing because the apparent Lugeon value is really just an index, not a true measure of the Lugeon value.

18.4 PREDICTION OF GROUT TAKES

For contractual and cost estimating reasons it is necessary to estimate grout takes, i.e. the volume of grout (or dry weight of cement) which will be absorbed by the foundation during the grouting operation.

This is difficult to do with any degree of accuracy because the penetration of the grout is dependent on fracture aperture, roughness, continuity and interaction with other sets of fractures and grout viscosity, pressure, duration etc.

Bozovic (1985) in his general report of the ICOLD Congress concludes that the correlation between grout take and Lugeon value is very weak. He suggested that considering the different rheological properties of grout and water, a correlation cannot physically exist. Figure 18.29 shows some data from Ewart (1985) which shows the poor correlation. Ewart (1985) presents similar data for Aabach Dam, as do Sims and Rainey (1985) for Gitaru Dam.

The authors agree that, if grout take and Lugeon value are compared directly, there is a poor correlation. However they are of the opinion that, if joint spacing is used to estimate fracture openings and grout penetration is estimated from grout particle size, grout pressure, viscosity and time, it should be possible to obtain a better prediction of grout take. It is not expected that an accurate prediction will result, only an improvement in the ability to predict. If this approach is coupled with grouting trials on the dam foundation it should be possible to achieve reasonable accuracy.

It is concluded that the best approach is to gather data from dam sites in similar geological environments and make initial estimates of take per metre of grout hole on this basis. For any reasonable degree of accuracy trial grouting on a representative part of the dam foundation will be required, with careful monitoring of takes in primary,

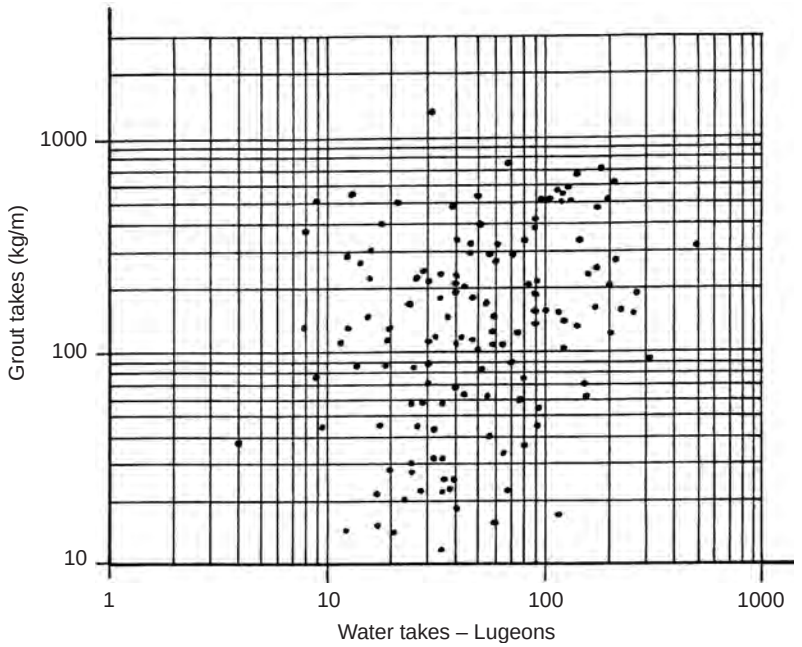


Figure 18.29 Grout take vs Lugeon value (Jawantzky, in Ewart, 1985).

secondary, tertiary holes etc. However unless proper attention is paid to the equipment used and selection of the area to be grouted, the results can be misleading.

18.5 DURABILITY OF CEMENT GROUT CURTAINS

From the authors' combined direct knowledge, which goes back to the early 1950s, cement grout curtains of dam that have operated for 50 years or so seem to have done their job admirably. Nevertheless there is evidence to suggest that seepage water will, albeit at very slow rates, gradually leach some of the compounds from the hardened grout. At Thomson (Victoria) and Corin (ACT) dams, there are calcium ions (along with magnesium and sulphate ions) in the seepage water at the dams' toes, but only traces of calcium ions in the storage water. For many dams this very slow deterioration probably would have little impact. The grouting will have done its main job of sealing up the large defects and the slow loss of the grout will not result in much of a change in the overall permeability of the rock mass.

Loss of compounds from the hardened grout probably begin with the slow loss of excess lime (CaO_2) that dissolves in the seepage water as calcium hydroxide ($Ca(OH)_2$). On contact with air (either direct contact or from dissolved air in, say, rain water), calcium carbonate ($CaCO_3$) forms. Depending on the flow conditions, the $CaCO_3$ may come out of solution. Another mechanism would begin with the solution in water of

carbon dioxide (CO_2) from bacteria in the ground. The resulting weak acid (H_2CO_3), then combines with the excess lime to give $CaCO_3$.

One would assume that this solution process is not unlike the processes described in Chapter 3 for the gradual loss of carbonates from carbonate rocks. If a partly-grouted joint in the rock mass has some unfilled zones and water seeps along those open parts of the joints, grout exposed to the flow will begin to suffer some loss. The loss rates would be expected to be very slow, particularly if only effectively narrow joints were available for the seepage water. Even if the loss of material did allow the cement grout to disintegrate to some degree, some of the inert particles in the cement grout may be able to join up the open parts of these joints to help maintain a reasonably low permeability for the rock mass.

A process like the one just described seems to have occurred at Blowering dam, a 112 m high rockfill dam in the Snowy Mountains Scheme (personal communications with C. Houlsby in the early 1980s). At Blowering much of the grouting was probably done with 5:1 and 6:1 W/C ratio (by volume). The weak grout in the joints of the rock 'bleeds' significantly, leaving patches of weak, hardened cement grout and water. Ultimately on storage filling seepage water was able to 'attack' the weak grout patches and reduce the effectiveness of the grout curtain. Under the spillway $CaCO_3$ has deposited in the underflow drains and lowered the capacity of the drainage system. The actual change in total seepage at Blowering is unknown, as seepage under the dam is not measured, but the left abutment ridge grouted by a single line curtain has significant seepage flows through it.

Talbingo dam (Snowy Mountains Scheme – completed 1971) and Thomson dam (Victoria – completed 1984) are two well-monitored dams that have long seepage records. Both are 160 m high rockfill dams. The storage at Talbingo is held close to full all the time, while Thomson, a water supply storage, does vary slowly with time and weather conditions.

At Talbingo, some 14,000 40 kg bags of cement were used on the curtain and 12,000 bags on the blanket (over the full core contact area). Generally, a 3 to 1 by volume water/cement was used as a starting mix, but the mixes were sometimes thinned to 5 to 1 if the ground proved to be tight. The seepage has been steadily dropping since records were started in late 1982, nearly 11 years after the lake's first filling was completed. The seepage ranges from 16 l/s just after records began to 10 l/s in early 2004. These figures would suggest no deterioration in the curtain, but one should also consider that:

- The defects in the foundations were probably not great in number and the rock below the excavated foundation for the core was inherently tight to start with.
- There may have been some 'silting up' of the open joints in the rock mass under the upstream rockfill shoulder.

The grout volume in bags of cement was much higher at the Thomson main dam. A total of nearly 21,000 bags were put into the curtain and over 100,000 bags in the blanket. About 8–9% of the total went into the river bed and lower 20–30 m of the abutments, but over 50% was pumped into 70 m of the right abutment. The upper right abutment was formed by two intersecting landslides and the ground was open to up to 90 m below the original natural surface.

So far at Thomson the base seepage in the period from 1990 to late 1996, when the storage was at a continuously high level, was recorded at about 10 l/s. The estimated permeability of the foundation is 1-2 Lugeons; during the investigations 100 Lugeon values were common on the upper right abutment.

It is probably too early to come to any definite conclusions on the grout curtain at Thomson. To date there has been no measurable deterioration.

Another Snowy Mountains dam, Eucumbene, a 116 m high earthfill dam with limited rockfill shoulders, seems to have behaved similarly to Talbingo dam. Early records from 1958, first filling, to 1998 show that:

Storage level (RL m)	*Head on base of core (m)*	*Seepage (l/s)*
1110	60	13
1143	93	17
1160	110	8

The conclusion on the potential deterioration of the grout curtain would be the same as drawn from the Talbingo records, although the same two factors noted for Talbingo probably apply also to Eucumbene.

18.6 CHEMICAL GROUTS IN DAM ENGINEERING

18.6.1 Types of chemical grouts and their properties

There are two distinct types of chemical grouts:

- Group A – colloidal solutions or pre-polymers, e.g. silica gel, ligno chrome gel, tannins, organic or mineral colloids, polyurethane.
- Group B – pure solutions, e.g. acrylamide, phenoplast, aminoplast.

The hydraulic behaviour of these grout types is different:

- Group A – behave as Bingham fluids with an initial shear stress required to mobilize the grout (as for cement and cement/bentonite grouts).
- Group B – behave as Newtonian fluids – as for water but with a higher viscosity.

Figure 18.30 shows typical flow properties of grouts. Note that the viscosity measured (in centipoise) is the slope of the graph at any point; the apparent viscosity is the slope of the line passing through the point (e.g. Point A) and the zero point (rather like a secant modulus compared to a tangent modulus for true viscosity). The intercept of the curve on the shear stress axis is the 'yield stress' or 'yield value' (also called 'cohesion' by Lombardi, 1985, by analogy with the shear strengths of soils). The value quoted will depend on the shear rate and may be determined by extrapolation of the straight line portion of the curve as shown in Figure 18.30. Most methods of measuring will give the apparent viscosity, rather than the yield stress and true viscosity and

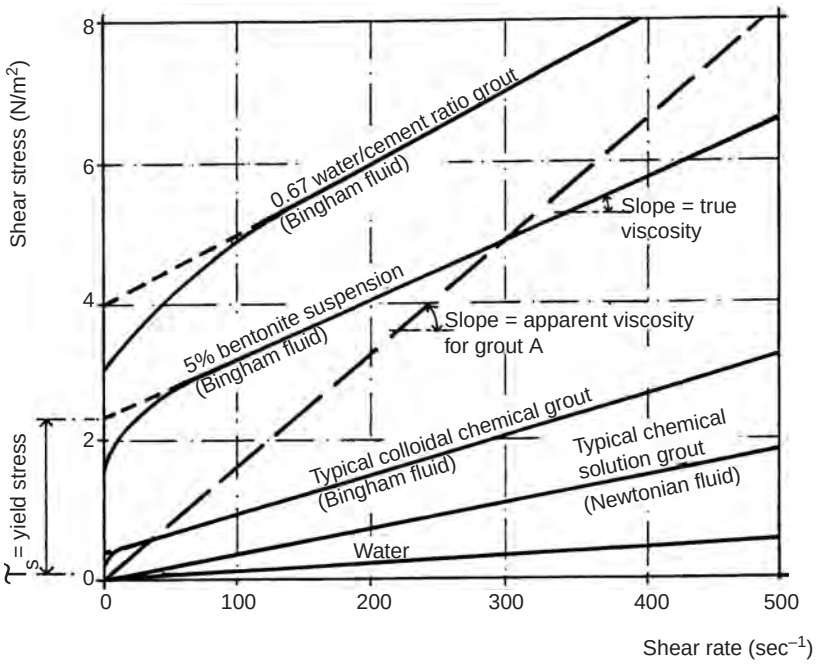


Figure 18.30 Typical flow properties for grouts (adapted from Littlejohn, 1985).

this must be taken into account. The authors' experience is that, with cement grouts, the method proposed by Nguyen and Bogor (1983, 1985), which uses a very sensitive vane shear type apparatus, is the most successful.

The viscosity is dependent on the concentration of the grout and the time after mixing. Figures 18.31 and 18.32 show these effects.

The gel time is dependent on time and temperature and on additives which are deliberately used to control it. This is discussed in Littlejohn (1985), Karol (1985), and Bruce *et al.* (1997).

There are many types of grout available and new products are coming onto the market all the time. Table 18.10 lists some of the more common grouts and their properties. Karol (1982a, b, 1983), Littlejohn (1985) and Bruce *et al.* (1997) discuss chemical properties of grouts in more detail.

18.6.2 Grout penetrability in soil and rock

Grout penetration is dependent upon:

(i) Whether penetration is by permeation (impregnation) in the voids in the soil or by causing hydraulic fracture by exceeding the in-situ horizontal stresses (0.5–2 times overburden pressure), or a combination of both. Australian and general overseas (except French) practice is to limit pressures to the permeation phase.

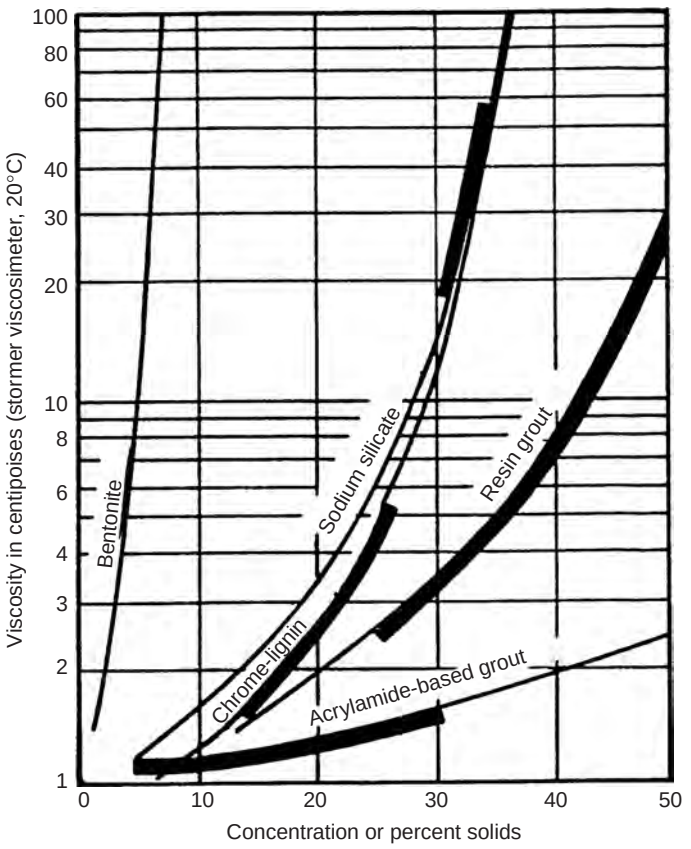


Figure 18.31 Viscosity (apparent) as a function of grout type and concentration (Karol, 1983).

(ii) The viscosity properties of the grout and then the pressure and time for which grouting proceeds.

For Bingham fluids, there is a limiting radius to which grout can be pumped because of the shear stress required to mobilize the grout. Littlejohn (1985) indicates that in soil this can be estimated from:

$$RL = \frac{\delta_w gHd}{4\tau s} + r \tag{18.3}$$

where RL = limiting radius of penetration; δ_w = density of water; g = acceleration due to gravity; H = hydraulic head; d = effective diameter of the average pore;

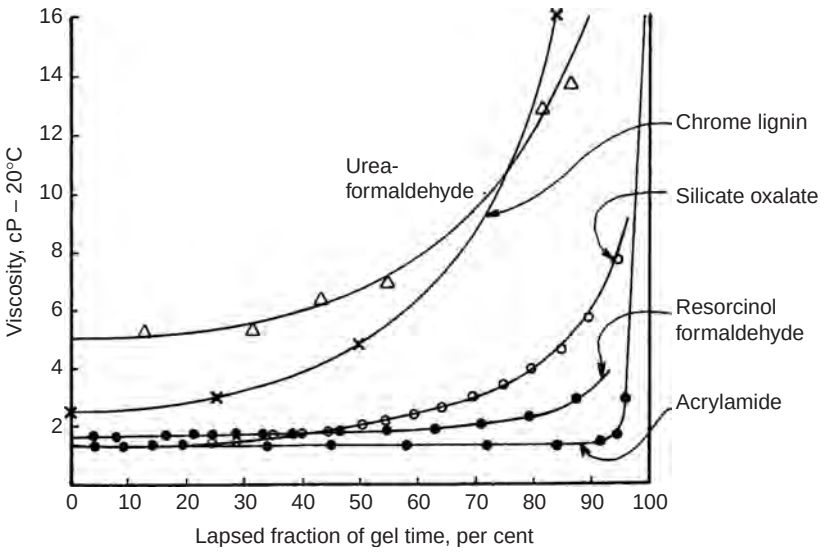


Figure 18.32 Viscosity (apparent) as a function of time (Littlejohn, 1985).

τs = Bingham yield stress; r = radius of spherical injection source = $\frac{1}{2}\sqrt{LD}$ where L = length of hole, D = diameter and:

$$d = 2\sqrt{\frac{8\mu k}{\delta_w g n}} \tag{18.4}$$

where μ = grout viscosity in centipoise; k = permeability of soil; n = porosity of soil.

In jointed rock, with a joint aperture of 2a, Lombardi (1985) indicates that the maximum radius is given by Equation 18.5.

This is equivalent to:

$$RL = \frac{Ha}{\tau s} \tag{18.5}$$

For Newtonian flow, in uniform isotropic soil from a spherical source, Littlejohn (1985) indicates that:

$$H = \frac{Q}{4nk}\left[\mu\left(\frac{1}{r} + \frac{1}{R}\right) + \frac{1}{R}\right] \tag{18.6}$$

where Q = flow rate at radius of penetration R and the time for grout to penetrate.

$$t = \frac{nr^2}{kH}\left[\frac{\mu}{3}\left(\frac{R^3}{r^3} - 1\right) - \left(\frac{\mu - 1}{2}\right)\left(\frac{R^2}{r^2} - 1\right)\right] \tag{18.7}$$

Table 18.10 Chemical grouts – a summary of properties.

Grout type	*Fluid behaviour*	*Typical viscosity centipoise*	*Gel time*	*Stability*	*Examples*	*Comments*
Sodium silicate	Newtonian initially, then Bingham	3–4 for permeability reduction 10 for strength. Viscosity increases as grout gels	30–60 minutes	1) Undergoes (loss of water and shrinkage syneresis on gelling. 2) Unstable in alkaline environment	Joosten process	Non toxic and not an environmental hazard. Syneresis less of a problem in finer soils (sand and silt). Limit is fine-medium sand. Krizak (1985) tests showed up to 10 or 100 times increase in permeability with time under high gradient seepage
Acrylamide	Newtonian	Less than 2	Gel time controlled by additives NaCl in groundwater may lessen gel time	Permanent	AM9 (now outdated because of neuro-toxicity). Injectite 80 (has higher viscosity)	AM9 was one of the most popular grout for a long time. Good permeability and permanence. AM9 was replaced by methyl acrylamides, but the toxicity of unreacted monomer remains
Phenoplast	Newtonian	1.5 to 3 initial and constant until gel starts	Gel time controlled by additives	Permanent except under alternating wet/dry conditions	e.g. Resorcinol plus formaldehyde + NaOH e.g. Geoseal	Medium toxicity. Geoseal 2–10 centipoise used for dams at Worsley (Brett, 1986)
Aminoplast	Newtonian	5, up to 10–20 with additives to stabilize gel time	Gel time controlled	Only gel in acid ($pH < 7$) environment. Permanent except in wet/dry conditions	Based on urea and formaldehyde	Toxic and corrosive prior to gelling (acid catalyst)
Acrylate polymer	Newtonian	2	Gel time controlled	Permanent	AC-400	Replaces AM9. Only 1% as toxic. Krizek and Perez (1985) tests showed it did not deteriorate with time under high (100) gradients.

Table 18.11 Grouting limits of common mixes (Littlejohn, 1985).

Type of Soils	*Coarse sands and gravels*	*Medium to fine sands*	*Silty or clayey sands, silts*
Soil characteristics			
Grain diameter	$D_{10} > 0.5$ mm	$0.02 < D_{10} < 0.5$ mm	$D_{10} < 0.02$ mm
Specific surface	$S < 100\ cm^{-1}$	$100\ cm^{-1} < S < 1000\ cm^{-1}$	$S > 1000\ cm^{-1}$
Permeability	$K > 10^{-3}$ m/s	$10^{-3} > K > 10^{-5}$ m/s	$K < 10^{-5}$ m/s
Type of mix	Bingham suspensions	Colloid solutions (gels)	Pure solutions (resins)
Consolidation grouting	Cement ($K > 10^{-2}$ m/s) aerated mix	Hard silica gels double shot Joosten (for $K > 10^{-4}$ m/s) Single shot: Carongel, Givanol, Siroc	Aminoplastic, phenoplastic
Impermeability grouting	Aerated mix, bentonite gel, clay gel, clay/cement	Bentonite gel, lignochromate, light Carongel, soft silicagel, vulcanisable oils, others (Terranier)	

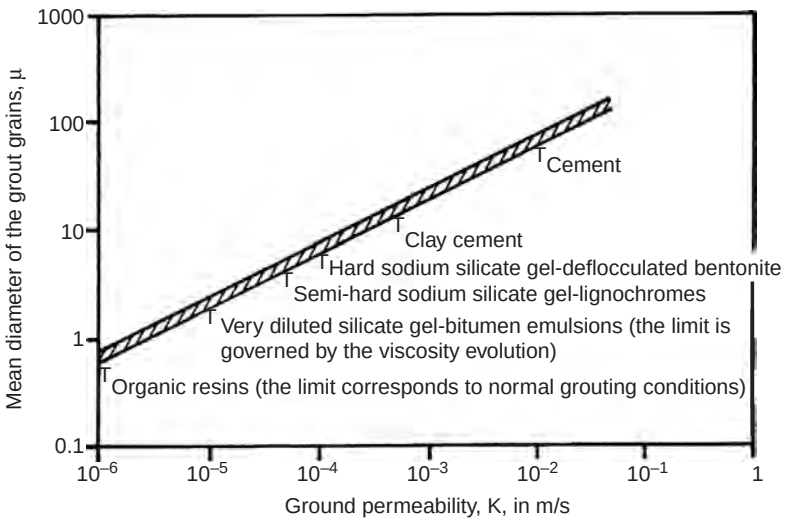


Figure 18.33 Limits of injectability of grouts based on the permeability of sands and gravels (Littlejohn, 1985).

i.e. time is proportional to R^3. This dictates that relatively close hole spacings are used for economic grout times, e.g. 0.5 m to 2.5 m.

There are different tables and graphs indicating the types of soils and soil permeability that can economically be grouted, e.g. Littlejohn (1985) suggests Table 18.11 and Figure 18.33.

Karol (1985) suggests that, based on a review of the literature, Figure 18.34 is a conservative assessment of groutability by permeation, i.e. without fracturing.

Caron (1982) suggests Table 18.12 as a basis for determining whether grouting is practicable, depending on ground conditions and the type of grout.

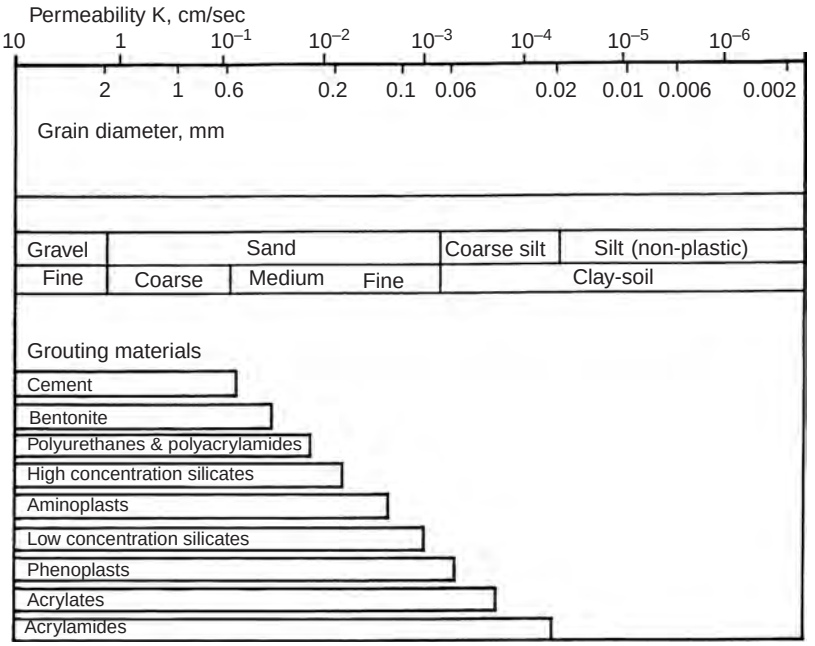


Figure 18.34 Limits of groutability (based on Karol, 1985).

Table 18.12 Limits of grouting (Caron, 1982).

	Type of ground			
	Coherent fissured soils		*Loose granular soils*	
Type of grouting method	*Large medium fissures* $K > 5 \times 10^{-7}$ *m/sec*	*Very fine fissures* $K < 5 \times 10^{-7}$ *m/sec*	*Coarse and medium* $K > 10^{-3}$ *m/sec*	*Fine* $K < 10^{-3}$ *m/sec*
By fracturing	–	–	C	C
By impregnation	C	CG	C	CG
By fracturing	–	–	–	CG

C = cement based grout; CG = chemical grouting; – = no case for grouting.

It is hard to envisage a case where grouting of soils with a permeability around 10^{-7} m/sec would be warranted or would result in significant reduction in permeability.

18.6.3 Grouting technique

The basic approach to use of chemical grouts is similar to that for cement grouts:

- Grout is injected into holes under pressure.
- Holes are drilled and grouted in stages to achieve a desired Lugeon or permeability closure.

When grouting in rock in which the holes remain open and packers can be set, downstage grouting with packer would normally be adopted. However, in most chemical grouting operations, either extremely weathered rock (i.e. virtually soil properties) or soil is being grouted and it is necessary to support the hole from collapse and use different methods to inject the grout.

Figure 18.35 shows the *tube-à-manchette* ('pipe with sleeves') technique which is used in Europe (and Australia) for grouting with chemicals.

In this technique the hole is drilled and cased to its full depth, the hole filled with a cement/bentonite grout and the tube a manchette installed. This consists of a 40 mm to 60 mm diameter PVC tube with 6 mm diameter boreholes in the wall of the tube at 300 mm or 333 mm intervals. The holes are covered by a rubber sleeve.

Once installed, the casing is withdrawn and the grout allowed to 'set' to give a low strength, relatively brittle grout.

The grouting operation is carried out by lowering a double packer to isolate one set of outlet holes as shown in Figure 18.35, setting the packers, then applying the grout pressure. The pressure of the grout lifts the rubber sleeve, fractures the bentonite-cement grout and allows the chemical grout to penetrate into the soil or weathered rock.

Some idea of pre-grouting permeability can be obtained from the grout flow rate and pressure but, as explained by Caron (1982), this may not be accurate because of unquantifiable pressure losses through the fractures in the cement/bentonite grout.

Caron (1982) indicates that, when grouting in sands and gravels, practice differs from country to country. In USA and Japan, grouting is commonly from the bottom of the casing used to support the hole, with the casing gradually withdrawn. For a two-shot grout, such as sodium silicate/calcium chloride, the base grout (sodium silicate) is injected as the casing is lowered and the reactant grout (calcium chloride) as the casing is withdrawn (as shown in Figure 18.36). This method does allow checking of permeability before grouting (but only crudely out of the bottom of the casing) and gives poorer control than the *tube-à-manchette* system.

As shown in Figure 18.37, grouting should be carried out on a 'closure' basis in depth and plan and at least 3 lines of holes should be used to achieve this. Initial spacing should be 2 to 3 times the expected final spacing.

18.6.4 Applications to dam engineering

Chemical grouts are expensive compared to cement or cement bentonite grouts. As a result, the use of chemical grouts will be restricted to those cases where seepage is critical and not controllable by cement grouts or in cases of remedial works, particularly in alluvial (soil) foundations where cement grouts are not applicable, and where other methods of controlling seepage such as slurry trench or diaphragm cut-offs as discussed in Chapter 10 are not suitable or too expensive for the particular case being considered.

As in the use of cement grouts, there is a limit to the permeability which can be achieved by chemical grouting. Littlejohn (1985) suggests that practical minimum average permeabilities are:

- $\times 10^{-7}$ m/sec in coarse sands and gravels (and only after 'sophisticated chemicals, careful injection procedures and close supervision').
- $\times 10^{-8}$ m/sec in 'fissured' rock.

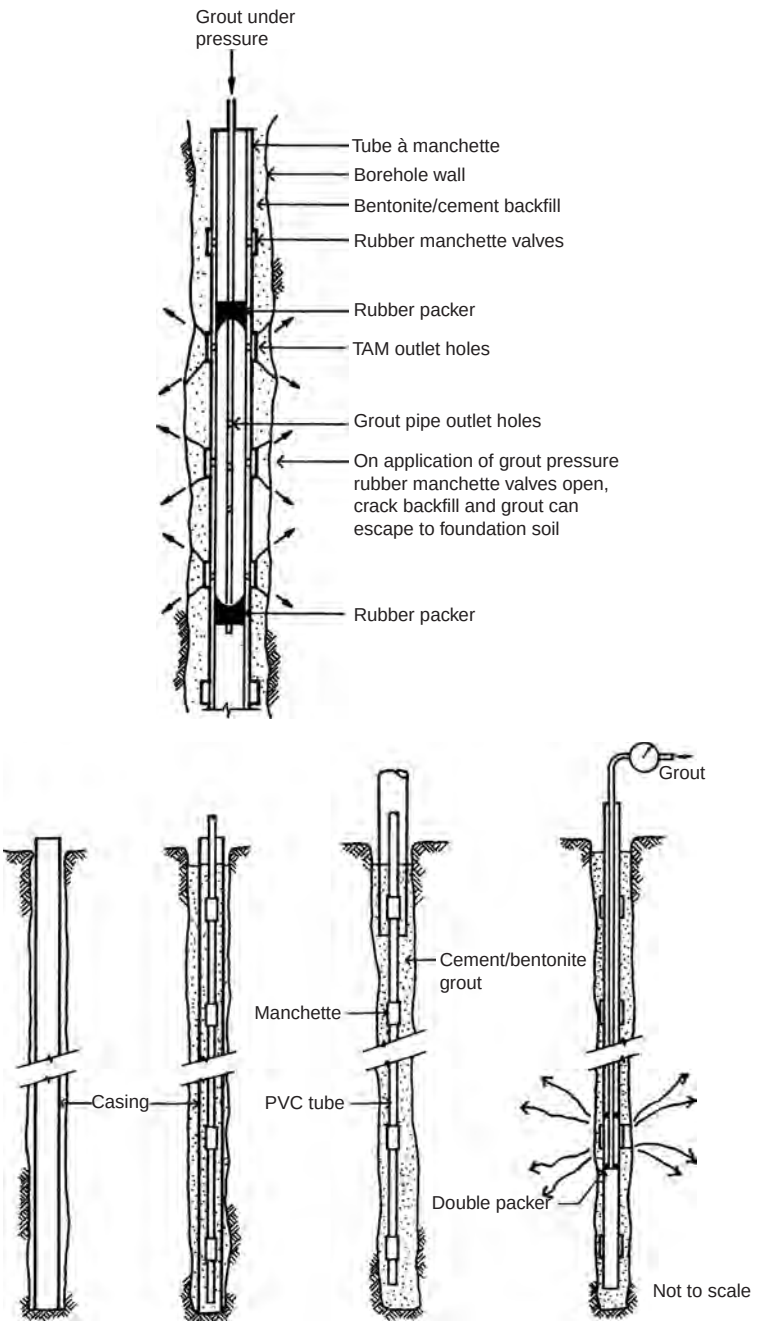


Figure 18.35 Tube-à-manchette grouting system (adapted from Brett, 1986).

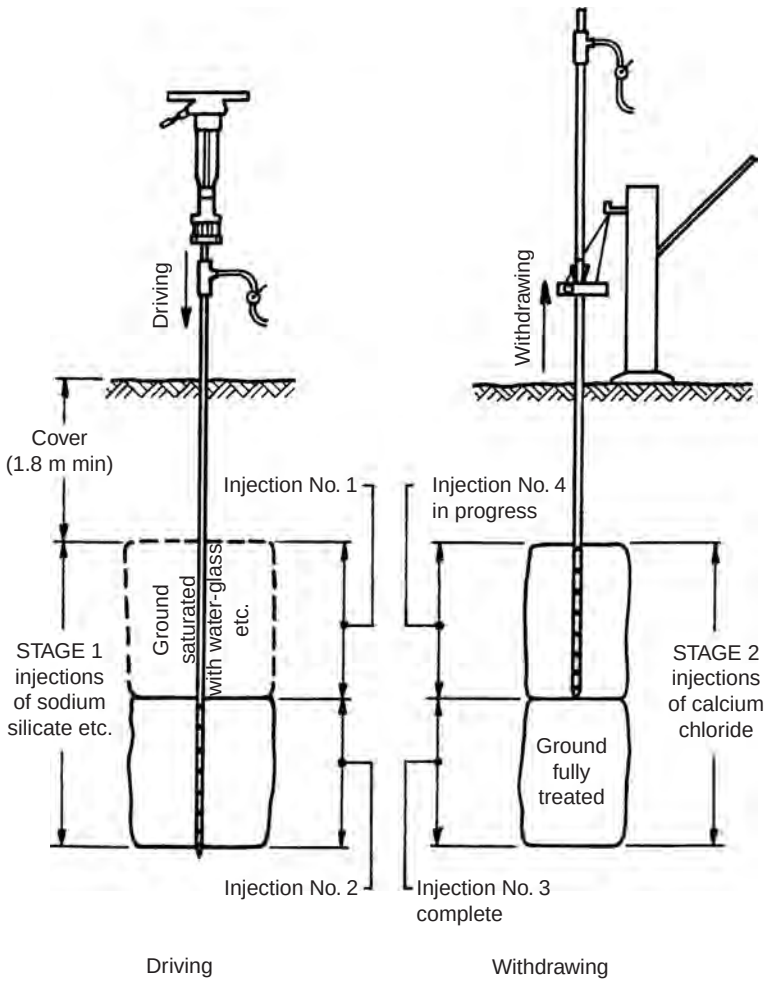


Figure 18.36 Open bottom hole method of grouting (Littlejohn, 1985).

Since the grout curtain width is likely to be narrow (say 1 m to 1.5 m maximum per row of holes), significant seepage reduction will result only when the original permeability is relatively high compared to the grout curtain (see Figure 18.20).

Brett (1986) presents a plausible argument for two or three stage cement/bentonite, followed by chemical grouting. The former is to fill the larger voids and hence reduce the cost of chemicals. Figure 18.38 illustrates this approach.

In many cases in the past where chemical grouts have been used to reduce leakage under dams on alluvial sand and gravel foundations, it would now be more reliable and less costly to use slurry trench or diaphragm wall cutoffs constructed of bentonite or bentonite-cement. These give a more controllable uniform width of low permeability cutoff than can be achieved by grouting and would generally be the authors' preference (see Chapter 10 for more discussion on this matter).

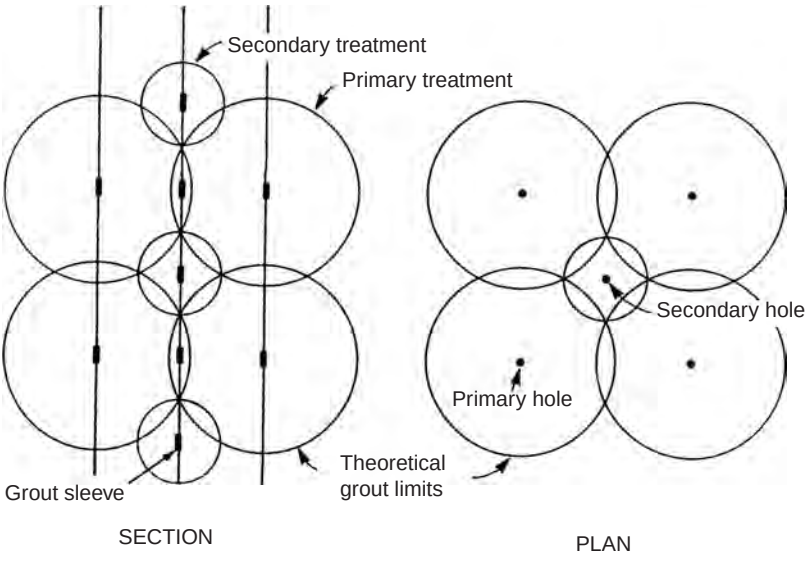


Figure 18.37 Staging of chemical grouting (Littlejohn, 1985).

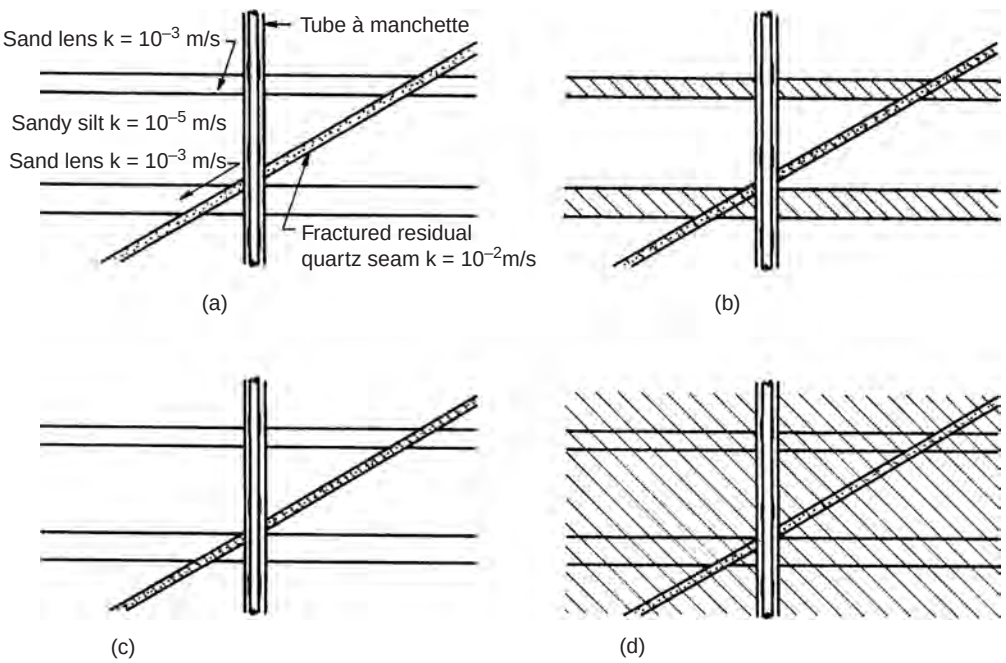


Figure 18.38 Multiple grout system (Brett, 1986): (a) Ground properties, (b) First stage cement grout, (c) Second stage bentonite grout, (d) Third stage chemical grout.

It should be noted that none of the authors have ever used chemical grouts in dam foundations and do not expect to.

Examples of the use of chemical grouting in dams are given in Littlejohn (1985), Brett and Osborne (1984), Davidson and Perez (1982) and Graf *et al.* (1985).

It will be clear from the above discussion that chemical grouting, although feasible for some jobs, should be viewed with some caution. Most chemical grouts are potentially toxic to some degree, some to the point of being of serious concern. Unless chemical grouting can be justified on cost grounds and is environmentally safe to use in the particular case under consideration, the authors would advocate not using them.

Like epoxy compounds and some other newly-developed compounds, chemical grouts are not the answer to all problems. If not mixed correctly or not applied correctly, they may cause more problems than they solve. The fourth author spent 2 years proving an ill-conceived grouting job done with chemical grouts was safe for the public; he is now convinced the chemical grouting did nothing to improve the situation, making the whole exercise an avoidable one if careful thought had been applied at the start of the design phase, especially to increased use of drainage provisions.

Interestingly, at the end of this job, some excess chemical grout ingredients were made available to a job at a power station that involved leakage around the inlet pipes. The outcome was a failure, and in hindsight, the chemical grout was the wrong material to use. The set chemical grout simply blew out from the gaps around the pipe. Worse still, when a further attempt was made to use cement grout yet again, a bond could not be established in the gap because of the film of chemical grout left behind, so the new repair failed as well. Such is evidence again of the incorrect use of new products and also the lack of attention paid at all stages of the proposed repair. New products are not necessarily the answer to everything, rather those old, forgiving products like cement, sand and water do have their place.

Chapter 19

Mine and industrial tailings dams

19.1 GENERAL

This chapter is intended to give an overall appreciation of mine and industrial tailings disposal practice. It is written for those who have not had extensive experience in tailings disposal and serves as an introduction to the subject, rather than forming a complete basis for design of a tailings disposal system.

The authors' experience is that the many mine tailings are disposed into reasonably conventional dams and the principles of dam engineering covered in the rest of this book apply. Some tailings are disposed of using the tailings themselves to construct the embankment e.g. using upstream, centreline or downstream methods, and this requires special design and construction features (see Section 19.5). More recently, thickened discharge, co-disposal and paste technology has been developed which is applicable in some situations. These methods are described briefly. This chapter mostly concentrates on the understanding of what tailings are and their properties, the methods of discharge, prediction of deposited density, the methods of disposal, estimation of seepage from tailings disposal areas and the particular aspects of slope stability, internal erosion and piping and performance under earthquake all of which are important in tailing disposal.

19.2 TAILINGS AND THEIR PROPERTIES

19.2.1 What are mine tailings?

Mine tailings are the end product of mining and mineral processing, after the mineral has been extracted, or the unwanted material separated, e.g. shale and clays from coal; clayey fines from iron ore or bauxite. As shown in Figure 19.1, the process will often involve crushing and grinding, leaching or separation, followed by dewatering or thickening before discharge to the tailings disposal area as a slurry. For coal and bauxite, the ore will not be ground and the process is one of separation of the other rock particles from the coal, or finer clays and silts from the bauxite gravel by washing and separation.

The thickening process may be assisted by addition of flocculants and/or other chemicals, e.g. polyelectrolytes such as the Magnafloc range, and inorganic salts such as gypsum. The polyelectrolytes are generally high molecular weight polyacrylamides,

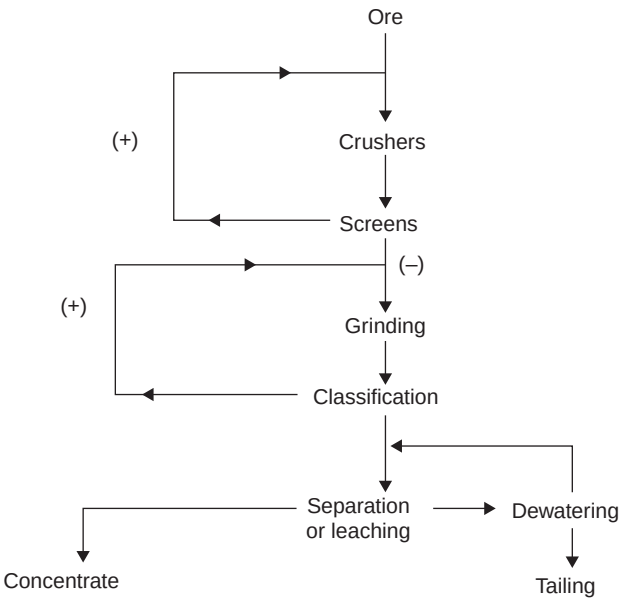


Figure 19.1 Procedures in mineral processing.

which have positive (cationic) or negative (anionic) charges. These adsorb to the electrically charged particles in the tailings and form large flocs which settle more quickly in the thickener. Inorganic salts such as gypsum ($CaSO_4$) operate by cation exchange, with high charge density Ca^{++} cations replacing low charge density Na^{+} cations, reducing dispersion effects and promoting flocculation as discussed in Chapter 7.

These chemicals affect the properties of the tailings discharged to the tailings disposal area and must be included in trial processes when testing tailings properties.

Dewatering is usually carried out in a thickener, with the dewatered tailings being recovered from the discharge cone as shown in Figure 19.2.

19.2.2 Tailings terminology and definitions

Most properties of tailings are described in soil mechanics terminology, but there are some terms in common usage which originate from mineral processing. Tailings are a mixture of solids and water (and air when not saturated) as shown in Figure 19.3, and the various terms used relate largely to the relative proportions of those present.

Commonly used terms and their definitions include:

Void ratio:

$$e = \frac{V_V}{V_S} \tag{19.1}$$

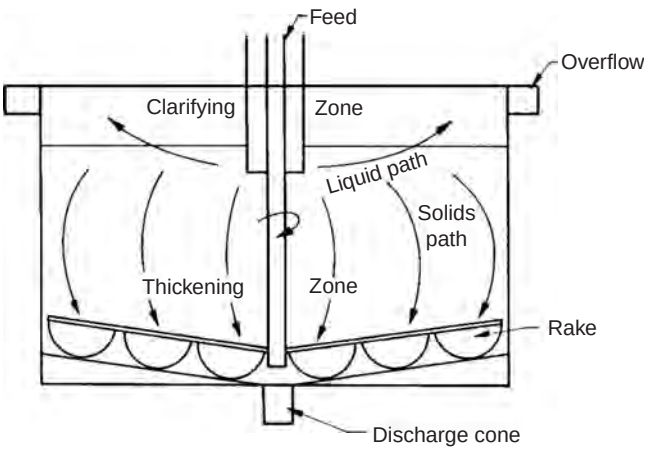


Figure 19.2 Minerals processing thickener.

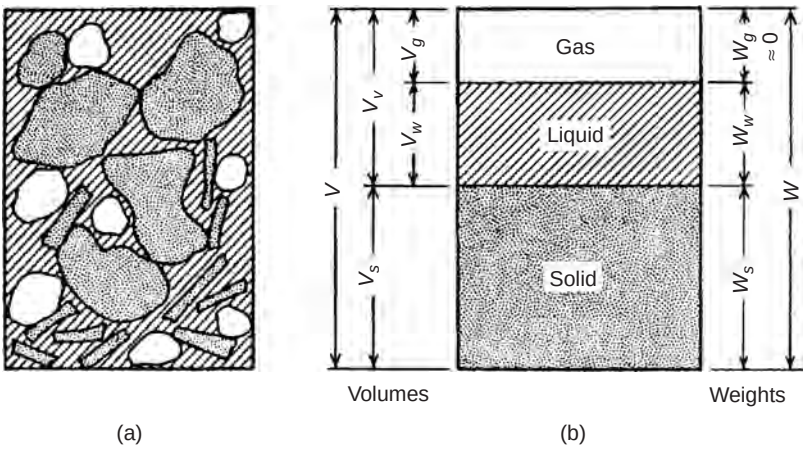


Figure 19.3 Tailings phase relationship, (a) element of tailings with air filled and water filled voids; (b) element separated into air, liquid and solid (Lambe and Whitman, 1981).

Degree of saturation:

$$S = \frac{V_W}{V_V} \tag{19.2}$$

Porosity:

$$n = \frac{V_V}{V} = \frac{V_V}{V_V + V_S} \tag{19.3}$$

Specific gravity (solids):

$$G = \frac{\gamma_S}{\gamma_W} \tag{19.4}$$

where V_V = volume of voids = $V_a + V_W$ (m^3); V_a = Volume of air in voids (m^3); V_S = volume of water (m^3); V_S = volume of solids (m^3); W_w = weight of water (kN); W_S = weight of solids (kN); M_W = mass of water (tonnes); M_S = mass of solids (tonnes).

$$\gamma_S = \text{unit weight of solids} = \frac{W_S}{V_S}\ (\text{kN/m}^3) \tag{19.5}$$

$$\gamma_W = \text{unit weight of water} = \frac{W_W}{V_V}\ (\text{kN/m}^3) \tag{19.6}$$

Water content (%):

$$w = \frac{W_W}{W_S}100(\%) \tag{19.7}$$

(May also be defined as $W_W/(W_W + W_S)$ by some metallurgists).

% Solids:

$$P = \frac{W_S 100}{W_S + W_W} = \frac{1}{1+w}\ (\%) \tag{19.8}$$

Degree of saturation:

$$S = \frac{V_W 100}{V_V} = \frac{\gamma_d\, w\, G\, 100}{G\gamma_w - \gamma_d}\ (\%) \tag{19.9}$$

Total unit weight:

$$\gamma_t = \frac{W_S + W_W}{V_V + V_S} = \frac{G + Se}{1+e}\gamma_W\ (\text{kN/m}^3) \tag{19.10}$$

Dry unit weight:

$$\gamma_d = \frac{W_S}{V_V + V_S} = \frac{G}{1+e}\gamma_W = \frac{\gamma_t}{1+w}\ (\text{kN/m}^3) \tag{19.11}$$

19.2.3 Tailings properties

19.2.3.1 *General*

Tailings properties differ considerably depending on the ore from which they are derived, the mineral process and whether the ore is oxidised (i.e. from weathered rock). Table 19.1 gives general characteristics of engineering behaviour.

When planning any tailings disposal project, tailings representative of the operational process plant should be tested to determine the properties required to predict behaviour in the storage. It should be noted that, in many cases, tailings produced from trial crushing and grinding in laboratory processing studies often are not truly representative of those produced by the operational process plant. This can lead to the perception that laboratory tests do not accurately predict field behaviour. The

Table 19.1 Summary of typical physical tailings characteristics.

Type of tailings	*General characteristics*
Ultra-fine tailings, phosphatic clays, alumina red mud	Clay and silt, high plasticity, very low density[3] and permeability
Washery tailings, coal, bauxite some iron and nickel ores	Clay and silt, medium to high plasticity, medium to low density and permeability
'Oxidised'[1] mineral tailings, gold, copper, lead, zinc, etc.	Silt and clay, some sand, low to medium plasticity, medium density and permeability
'Hard rock'[2] mineral tailings, gold, copper, lead, zinc, etc.	Silt and some sand, non plastic, high density, medium to high permeability

Notes: (1) Ore is completely or highly weathered, or altered rock.
(2) Ore is slightly weathered to fresh rock.
(3) Assuming no desiccation

authors' experience is that laboratory tests on representative samples can reasonably predict field behaviour.

However it is always important to build in sufficient flexibility in the design of a tailings storage facility to accommodate tailings properties may differ from the ones envisaged in the design phase.

19.2.3.2 *Particle size*

Figure 19.4 gives some examples of particle size distributions from washery, oxidized and hard rock tailings.

The particle size distribution for oxidised tailings and tailings from washeries (e.g. coal, iron ore, bauxite) depends on the test method used and particularly on whether dispersants are added.

The standard soil mechanics test for determination of particle size of fine materials is to use a hydrometer analysis with a dispersant (calgon, i.e. sodium hexametaphosphate) added to break the particles to their constituent size. In the thickener, and as transported to the tailings disposal area, the particles may remain flocculated. Figure 19.5 shows particle size distributions for tailings from two coal mines (Wambo, Hunter Valley; Riverside, Central Queensland); a bauxite mine (Weipa, North Queensland) and iron ore mines (Newman and Hamersley, Western Australia). Tests with and without dispersant show that the 'true' behaviour is to act as a silt-sand mix, whereas with dispersant added the tailings have high clay size fractions.

19.2.3.3 *Mineralogy*

The tailings' behaviour can often be related to the mineralogy of the constituent particles. To illustrate this, Tables 19.2 and 19.3 list mineralogy, chemistry and soil mechanics properties of several tailings. The following should be noted:

– The bauxite tailings from Weipa have a high proportion of amorphous clay minerals which, while fine grained, lead to a relatively low liquid limit and plasticity index and give favourable (for fine tailings) sedimentation characteristics.

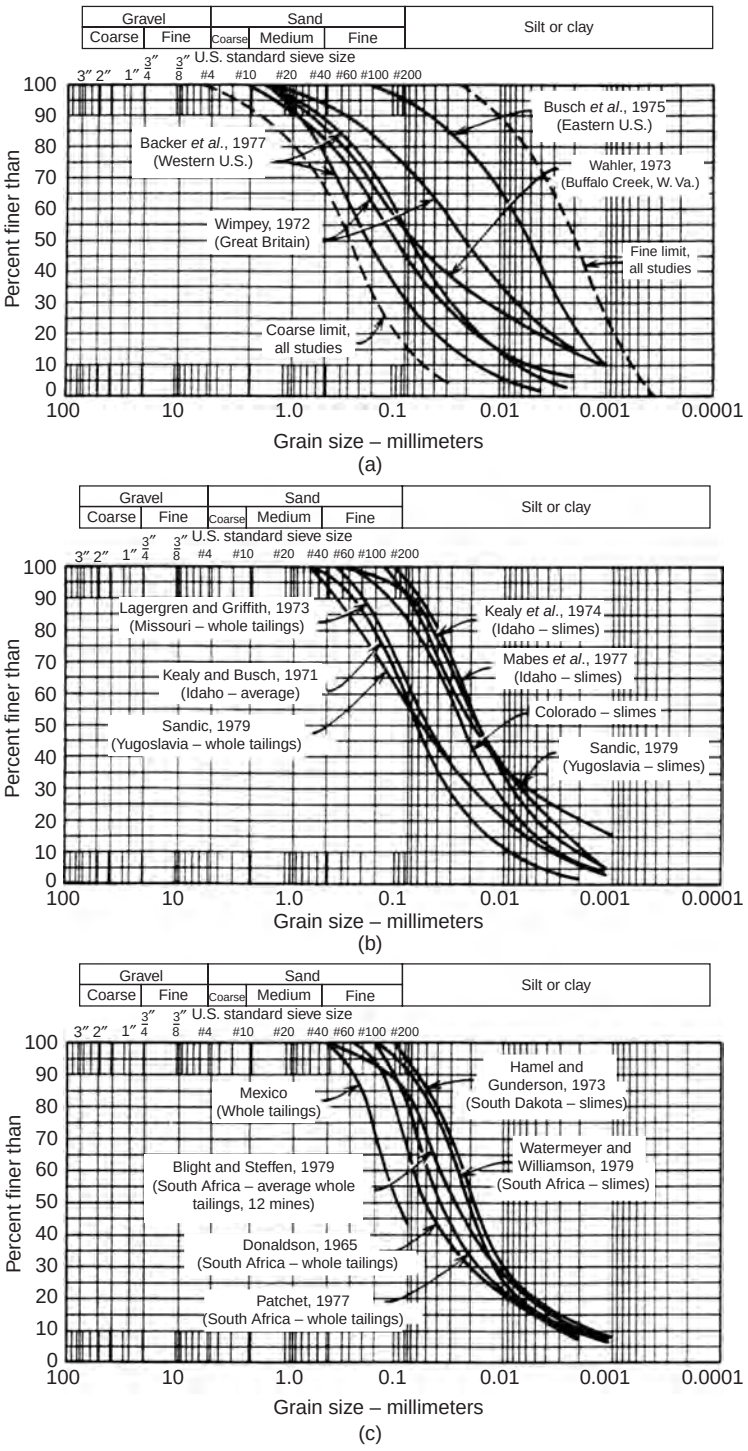


Figure 19.4 Examples of tailings particle size distribution: (a) coal washing; (b) lead-zinc; (c) gold-silver.

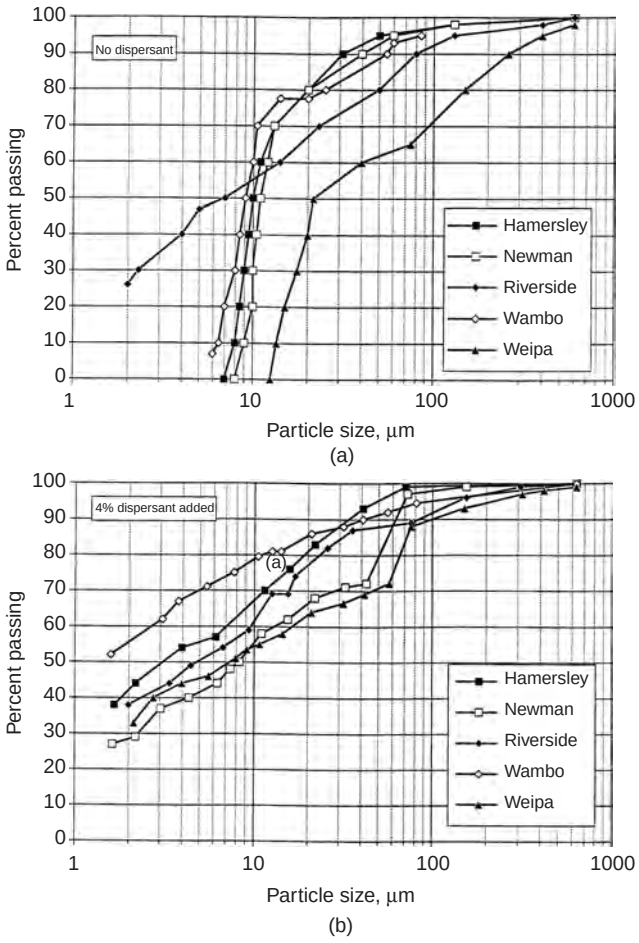


Figure 19.5 Particle size distributions of coal, iron ore and bauxite washery tailings (a) without; (b) with dispersant added.

- The coal washery tailings from Wambo have a high proportion of sodium montmorillonite, which leads to a very low settled density.
- The coal washery tailings from Riverside and gold tailings from North Kalgurli, contain clays which have intermediate activity between Weipa and Wambo and behave accordingly. The coal content of Wambo and Riverside tailings give a low specific gravity.

19.2.3.4 *Dry density and void ratio*

The iron ore tailings from Newman and Hamersley have a significant haematite content, giving a high specific gravity. The clay minerals are not very active and, although fine grained, the tailings tend to sediment favourably. The Broken Hill tailings have

Table 19.2 Mineralogy and chemistry of some fine grained tailings.

Tailings	*Mineralogy*	*Chemistry (% by weight of dried tailings)*								*PH tailings water*
		SiO_2	Al_2O_3	Fe_2O_3	*CaO*	*MgO*	Na_2O	K_2O	*LOI (1)*	
Weipa 1	Gibbsite (45%), boehmite (18%), kaolin (12%), quartz (17%), hematite (5%)	19	52	6	<0.1	<0.1	<0.1	<0.1	18	6.9
Weipa 2	Gibbsite (34%), boehmite (35%), kaolin (10%), quartz (8%)	10	53	12	<0.1	<0.1	<0.1	0.1	19	6.4
Wambo	Na and Ca montmorillonite, kaolin, coal, quartz (2)	28	12	1	0.9	1.0	0.8	0.4	51	6.8
Riverside	Illite, Ca montmorillonite, kaolin, quartz (2)	29	13	0.5	<0.1	0.3	0.3	0.7	54	7.4
Mt Newman	Hematite (40%), kaolinitic shale (50%), quartz (6%) (2)	17	12	63	0.4	0.7	<0.1	0.4	5	7.4
Hamersley	Hematite (34%), kaolinitic shales (52%), limonite (8%), goethite (19%) (2)	20	14	60	0.1	0.3	<0.1	0.1	7	7.1
North Kalgurli	Quartz, kaolin, gibbsite (2), host rock, dolerite, basalt, sulphide and oxide ores	55	14	13	2.3	1.1	2.8	2.2	7	7.9
Broken Hill	Quartz (50%), rhodenite (20%), calcite (12%), garnet (6%), manganhedbergite (4%), pyrrhotite (4%)	56	7	12	6.7	1.1	1.1	1.3	1	7.7

Notes: (1) Loss on ignition (LOI) at 1000°C.
(2) mineralogy percentages are approximate and vary with production sources.

Table 19.3 Soil mechanics properties of some fine grained tailings.

Tailings	*Water content (%)*	*Dry density (t/m^3)*	*% solids*	*Soil particle density (t/m^3)*	*Atterberg limits (%)*			*Particle size(%)*		
					Liquid limit	*Plastic limit*	*Plasticity index*	*Sand*	*Silt*	*Clay (2)*
Weipa 1	675	0.14	13	2.75	44	27	17	20	47	33
Weipa 2	362	0.25	22	2.85	43	26	17	15	45	40
Wambo	411	0.22	20	1.86	74	28	46	14	36	50
Riverside	250	0.32	29	1.74	44	28	16	16	50	34
Newman	192	0.46	34	3.70	33	22	11	20	60	20
Hamersley	169	0.50	37	3.50	30	21	9	5	55	40
North Kalgurli	71	0.94	59	2.81	28	21	7		Not	
Broken Hill	462	0.20	18	3.05		Non plastic			available	

Notes: (1) Particle size with dispersant added as per AS1289.
(2) Percentage finer than 0.002 mm.

Table 19.4 Typical in place densities and void ratio for non-desiccated tailings (adapted from Vick 1983).

Tailings type	*Specific gravity*	*Void ratio*	*Dry density (tonnes/m^3)*
Fine coal refuse			
Eastern US	1.5–1.8	0.8–1.1	0.7–0.9
Western US	1.4–1.6	0.6–1.0	0.7–1.1
Great Britain	1.6–2.1	0.5–1.0	0.9–1.35
Oil sands			
Sands	–	0.9	1.4
Slimes	–	6.0–10.0	
Lead–zinc Slimes(1)	2.9–3.0	0.6–1.0	1.5–1.8
Gold–silver Slimes	–	1.1–1.2	
Molybdenum Sands	2.7–2.8	0.7–0.9	1.45–1.5
Copper Sands	2.6–2.8	0.9–1.4	1.1–1.45
Taconite Sands	3.0	0.7	1.75
Taconite Slimes	3.1	1.1	1.5
Phosphate Slimes	2.5–2.8	11.0	0.25
Gypsum treated tailings	2.4	0.7–1.5	
Bauxite Slimes	2.8-3.3	8.0	0.3(2)

Notes: (1) For hard rock tailings; (2) Low by Australian standards 0.5–0.9 t/m^3 more likely.

been produced from high strength unweathered rock and yield a non plastic silt-sand mix which settles rapidly to a relatively high density. Table 19.4 gives some typical in-place dry densities and void ratios. The lower void ratios/higher densities correspond to greater depths within a deposit and would be typical of tailings which have been placed into a water pond, i.e. sub-aqueous placement. If desiccation of the tailings occurs (i.e. drying in the sun), dry densities would usually be 20% to 50% higher than those shown.

The in-place density also depends on the thickness of tailings (the thicker the tailings the higher the average density) and whether the tailings drain vertically towards the foundation, with higher effective stresses and higher densities when consolidation is complete.

General trends are better shown by the void ratio because the dry density is significantly affected by the specific gravity of the particles. Vick (1983) indicates that most high strength rock (and even low strength rock) tailings will have in-place void ratio of 0.6 to 0.9 for sand sized particles and 0.7 to 1.3 for slimes. The exceptions are phosphatic clays, bauxite (alumina) and oil sands and slimes.

It will be noted that the dry densities are very much lower than the specific gravity (or dry density) of the parent rock. In many mining operations a very small percentage of the ore is removed as mineral in the milling process and it would not be unusual for the tailings volume to be 150% to 200% of the original volume of the mined ore.

The relative density of the silt and sand sized tailings has an important influence on the potential for liquefaction. The relative density of spigotted tailings sands above water, i.e. in the beach zone, can be expected to be in the range 30% to 50% (Vick, 1983). Morgenstern and Kupper (1988) indicate that relative densities of 50% to

Table 19.5 Typical tailings permeability (adapted from Vick, 1983).

Type	*Average Permeability m/sec*
Clean coarse, or cycloned sands with less than 15% fines	10^{-4} to 10^{-5}
Peripheral-discharged beach sands with up to 30% fines	10^{-5} to 5×10^{-6}
Non plastic or low-plasticity slimes	10^{-7} to 5×10^{-9}
High plasticity slimes	10^{-6} to 10^{-10}

Note: These are likely to be representative of average vertical permeabilities.

65% can be expected for sands placed above water and 35% to 40% when placed underwater. De Groot *et al.* (1988) show relative densities from 10% to 50% (mainly 25% to 40%) for sands placed below water. Thus silt/sand tailings placed below water are likely to be susceptible to liquefaction under earthquake and static shear.

19.2.3.5 *Permeability*

The permeability of mine tailings depends on the type of tailings, particle size and mineralogy, deposition method, degree of consolidation and/or desiccation (dry density/void ratio), cracking or other structure and whether the tailings are saturated or partially saturated. When tailings are deposited unthickened, there is segregation of the coarser particles relative to the fines with distance from the discharge point so the tailings permeability will vary in the deposited tailings, typically higher near the spigot point, if the sands and coarse silts separate out, and lower distant from the discharge point where the fine tailings or 'slimes' deposit. It is also lower, deeper in the deposit than at the surface, due to consolidation.

In addition, within each layer of tailings, the larger particles settle at a different rate from the finer ones, so there is vertical segregation in the layers. For most tailings, the larger particles settle to the bottom of the layer and may form a thin sandy 'parting'. For coal washing tailings, the coarser particles are often coal, with a low specific gravity, and these rise to the top of the layer.

Table 19.5 gives some typical tailings permeabilities, which are probably representative of vertical permeabilities.

The horizontal permeability will change with the degree of segregation and variability of deposition. Near the discharge area (say the first 50–100 metres), it is likely the vertical segregation will give an equivalent horizontal permeability at least 10 times and probably 100 times the vertical. In the slimes the effect is likely to be small, with horizontal and vertical permeabilities similar.

19.2.3.6 *Properties of water in tailings*

The water (or liquor) accompanying tailings often contain dissolved salts, heavy metals and other residual chemicals from the mineralogical process.

The water may be highly acid or alkaline, have high salts content, heavy metals content or have arsenic or cyanide present. Each may have important implications

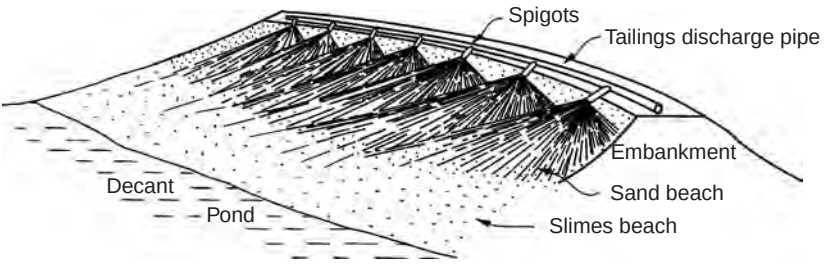


Figure 19.6a Tailings discharge by header and spigot.

on the impact of the tailings disposal area on the environment. Some common problems are:

- Gold tailings – cyanide.
- Coal tailings – high salts content, sometimes sulphides.
- Copper, lead-zinc, tin tailings – heavy metals, sulphides (which oxidise and lead to leaching of heavy metals and yield acid water).
- Alumina tailings – caustic soda (NaOH), unless the tailings are neutralized before discharge.
- Uranium tailings – heavy metals, radon.
- Other tailings may have quite acceptable water quality, e.g. some bauxite and iron ore washeries.

The science of predicting the chemistry of the tailings liquor and the fate of contaminants in the groundwater has advanced considerably over the last 20 years. It is a specialist area and the advice of a suitably qualified person should be sought.

19.3 METHODS OF TAILINGS DISCHARGE AND WATER RECOVERY

19.3.1 Tailings discharge

Most tailings are pumped to the tailings disposal area after thickening to a high water content (e.g. 35% to 40% solids content) slurry. In thickened tailings the solids content may be as high as 50% to 60%. The tailings are discharged into the storage from a single or several discharge points, or from spigots, as shown in Figures 19.6 and 19.7. A single fixed discharge point gives little flexibility in management of the tailings and is usually undesirable.

Spigotting facilitates the uniform spreading of the tailings and encourages desiccation. For conventionally non-thickened tailings spigotting encourages the segregation shown in Figure 19.6 with the sand and coarse silt settling near the embankment.

For some fine grained high clay content tailings, a 'spray bar' is attached to each spigot to further distribute the tailings and allow uniform beaching. The 'spray bar' consists of, for example, a 100 mm diameter pipe with 20 mm diameter holes at



Figure 19.6b Example of header and spigot deposition (Davidson and Winkler, 2014).

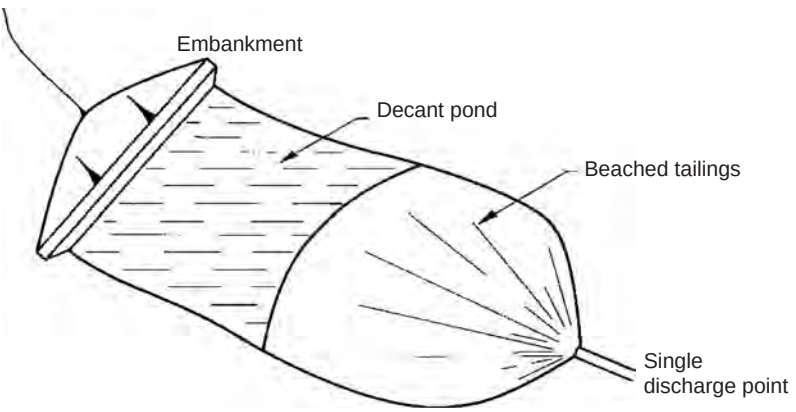


Figure 19.7 Tailings discharge from a single pipe.

500 mm spacing. The objective is to give a uniform deposition of tailings, allowing the greatest degree of sorting (or 'classifying') of the tailings away from the discharge points and to optimise desiccation.

It is usually preferable to discharge the tailings from the dam embankment, so that the tailings cover the storage area in the vicinity of the dam and potentially reduce seepage. If it is desired to blanket the base of the storage with tailings as shown in Figure 19.43(b), it is necessary to spigot progressively from all round the storage, moving the main discharge line up the slope as the tailings rise in the storage. In the case of upstream or centreline type construction (see Section 19.5), it is essential that the tailings are discharged from the embankment and the water pond kept small and well away from the edge of the embankment.

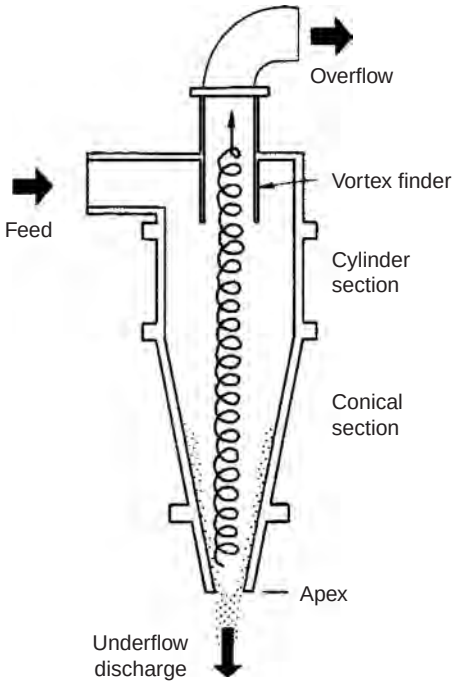


Figure 19.8 Typical cyclone for tailings.

19.3.2 Cyclones

When tailings dams are to be constructed of tailings by the upstream or centreline methods, it is common to separate the coarse fraction of the tailings from the fine fraction or 'slimes' by use of cyclones.

Figure 19.8 shows a typical cyclone. The 'whole' tailings are fed under pressure into the cylindrical cyclone. The coarser particles separate from the fines by centrifugal force (there are no moving parts) and spiral downward through the conical section as 'underflow'. The finer fractions and most of the water rise to the top outlet as 'overflow'.

The performance of a cyclone depends on many factors, including the particle size, specific gravity, clay content of tailings, size of cyclone, pressure etc. and advice should be sought from the cyclone manufacturer as to how effective they are likely to be.

The use of cyclones is discussed in Vick (1983) He indicates that reasonably clean sand can be obtained from tailings with less than 60% passing 0.075 mm, provided the tailings are essentially non plastic and free of clay minerals. Such sands will have high permeability and high shear strength.

Cycloning may be used in several ways:

– *Stationary cyclone plant* – a central high capacity cyclone station is established near the dam. Sands from the cyclone can be placed in the dam by conventional

earthmoving equipment, and may be compacted. This may facilitate use of tailings for dam construction in seismic areas.

– *On dam cycloning* – the most common technique. Several small cyclones are set up on the dam and the underflow discharged on to the embankment. The cyclones are moved as the dam is raised. Vick (1983) indicates that relative densities of 45% to 55% are likely to be achieved if no compaction is carried out.

At the El Cobre Tailings Dam in Chile, a large stationary cyclone plant was used, with the water content of the under flow engineered to give the required downstream slope of 4H:1V. This slope was continuously tracked by a small bulldozer to compact the tailings to a relative density of at least 65% (Cohen and Moenne, 1991). This material was not saturated, dense and as a result, non liquefiable.

Davidson and Winkler (2014) describes the options for cyclones for downstream and centreline construction using the tailings underflow.

19.3.3 Sub-aqueous vs sub-aerial deposition

If for climatic reasons, lack of a large discharge area, acid generation control, or other reasons; tailings are deposited under water, the method is known as sub-aqueous deposition. Fine, high clay content tailings deposited sub-aqueously will usually achieve a low settled density, very low shear strength and will be very compressible. Thus large, costly storages are required to contain the same quantity of tailings and long term rehabilitation is more difficult than if sub-aerial deposition is used, because the surface of the tailings is wet and low strength. If water cover is maintained on the tailings, seepage quantities from the storage will be relatively high.

The method where the tailings are discharged in a 'beach' above the water pond and where the deposition is cycled so that the tailings are allowed to dry by desiccation is known as sub-aerial deposition.

Desiccation induces negative pore pressures in the tailings, which results in consolidation of the tailings to higher densities than by subaqueous methods. The permeability and compressibility are also reduced, although cracking by desiccation may affect the permeability.

In the ideal situation where the climate, storage area and tailings permit, the tailings will remain in a partially saturated condition and result in significantly reduced seepage from the storage. Post placement consolidation may be very small and the strength such that access on to the surface for rehabilitation can be easy. Figure 19.9 shows a conceptual design of such a system (Robertson *et al.*, 1978).

Blight (1988) also discusses the benefits and disadvantages of desiccation. He points out the risks of planning to use the benefits and then not being able to achieve them. He cites an example where loss of control of water return resulted in the achieved dry density about half the expected. The authors' experience is that the technique can be successfully used, if an adequate area is provided to accommodate the rate of tailings production, the evaporation is sufficiently high and care is taken to operate the area properly. In such cases, even high clay content tailings have been desiccated sufficiently for use with upstream construction.

Table 19.6 summarizes some of the advantages and disadvantages of sub-aerial depositions.

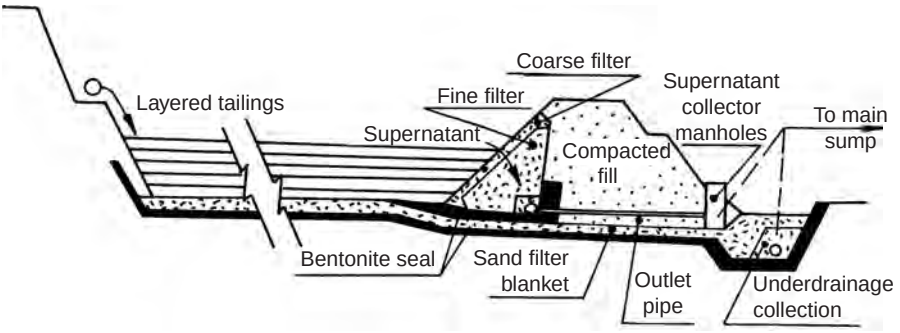


Figure 19.9 Sub-aerial placement of tailings (Robertson *et al.*, 1978).

Table 19.6 Sub-aerial disposal advantages and disadvantages (Knight and Haile, 1983; Lighthall, 1987; from Ritcey, 1989).

Advantages:

a) Higher unit weights are achieved in the tailings mass, resulting in better utilization of the storage facility.
b) The drained nature of the tailings and removal of surface ponding adjacent to embankments allows construction by upstream methods.
c) The low permeability, laminated structure of the tailings deposit reduces seepage.
d) At decommissioning, the tailings are fully drained and consolidated, allowing immediate construction of a surface seal and cover and elimination of any long term seepage.
e) The drained nature of the tailings increases resistance to liquefaction.
f) There is a low hydraulic head on the liners; therefore, a wide choice of liners is possible.
g) Capping and covering should be facilitated.

Disadvantages:

a) Dusting may occur unless wetted.
b) Method may not be suitable during extremely cold periods. Therefore the disposal area is divided into two, for alternative summer-winter deposition. Winter deposition is somewhat conventional (subaqueous).
c) Possibly higher initial cost to construct compared to other methods.
d) Requires a separate water storage reservoir which can be constructed for tailings economically water and runoff water recycling or discharge.

The authors have seen cases where increased production of tailings, poor control of return water, low winter evaporation, and a lack of consideration of the fact that early in a dam's life the surface area is often less than later when the tailings level has risen, have led to virtually no desiccation being achieved.

Blight (1988) also cautions against drying the tailings to the stage where they crack, as this may lead to a potential for piping failure. This would only be a problem in particular instances of upstream construction using small paddocks where the desiccation cracks could penetrate from the water to downstream, as is common in South Africa and sometimes in parts of Australia.

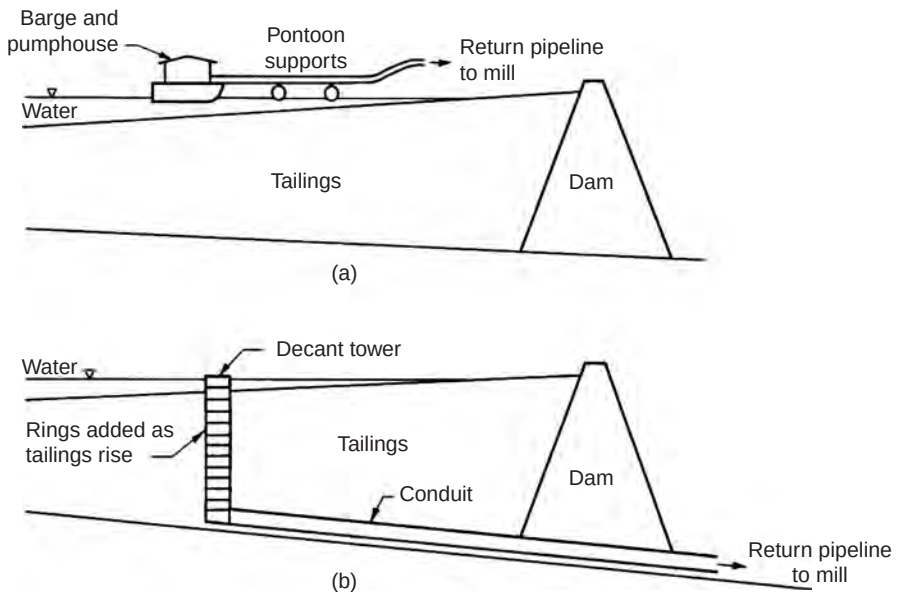


Figure 19.10 Water recovery methods.

In general, one will not aim for the 'full' sub-aerial deposition method shown in Figure 19.9. This was developed for uranium tailings, where requirements for seepage control are very stringent. The costs of underdrainage will not be warranted in most cases but may be required to collect contaminated seepage and to encourage consolidation of the tailings. One should, however, aim to obtain as much benefit as possible from desiccation for the reasons outlined above. It is important to consider the 'whole of life' of the facility because many of the benefits of sub-aerial deposition and desiccation are in reduced rehabilitation and long term environmental impacts.

19.3.4 Water recovery

Water is removed from the tailings storage as it 'bleeds' from the beached tailings or as tailings settle and consolidate if deposited sub-aqueously.

This is usually achieved by a pump mounted on a floating barge or a decant tower, as shown in Figure 19.10.

There is no preferred method each case has to be considered on its merits. The floating barge system is usually more flexible and requires less stringent control of tailings deposition to maintain the decant pond near the decant tower. It also avoids the potential weakness, from an internal erosion and piping viewpoint, of having a conduit through the embankment. It is usually desirable to keep the water surface area to a minimum to reduce evaporation of water (and hence reduce total water losses) and to optimise the benefits of desiccation of the beached tailings.

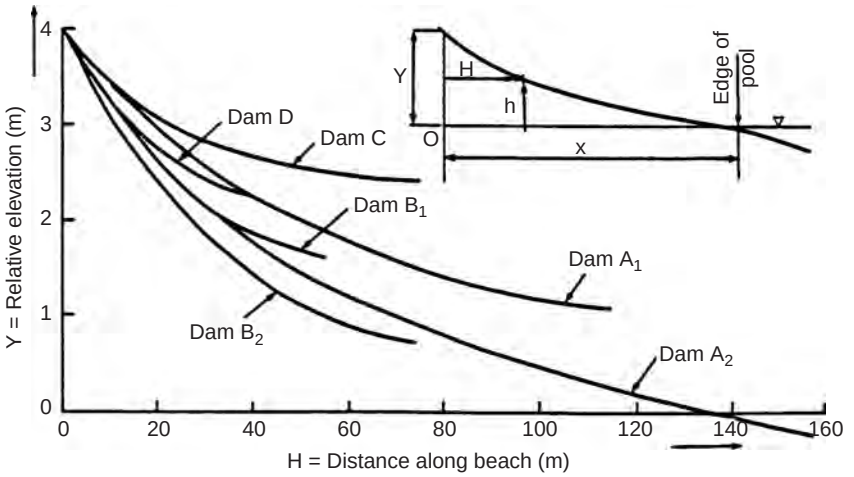


Figure 19.11 Measured beach profiles on six platinum tailings dams (Blight, 1988).

19.4 PREDICTION OF TAILINGS PROPERTIES

19.4.1 Beach slopes and slopes below water

Tailings deposited in a tailings dam form a beach above the water level and also fill the storage below the water level with a sloping surface. The beach above the water level is commonly steeper adjacent to the discharge point than further away.

Figure 19.11 shows observed beach profiles for several dams containing platinum tailings (Blight, 1988).

Figure 19.12 shows observed profiles for several tailings dams, in which fine grained tailings with high clay contents are stored. Properties for these tailings are given in Figure 19.5 and Tables 19.2 and 19.3.

The fact that the tailings deposit in this way is important because it results in a reduction in the available tailings storage volume compared to water storage, and determines the position of the water pond, which is important particularly for upstream construction.

Blight and his co-workers have investigated beach profiles by laboratory experiments in sloping flumes, and by observing beach profiles in the field. This work is described in Blight and Bentel (1983), Blight (1987) and Blight (1988). The concept they have developed was originally proposed by Mellent *et al.* (1973). It was found that by plotting dimensionless parameters h/y versus H/X, as shown in Figure 19.13, the different profiles in Figure 19.11 all plotted to give a single 'master' beach profile with the equation:

$$h/y = (Y/X)(1 - H/X)^n = i_{av}(1 - H/X)^n \qquad (19.12)$$

where i_{av} is the average slope, and n is determined for the tailings.

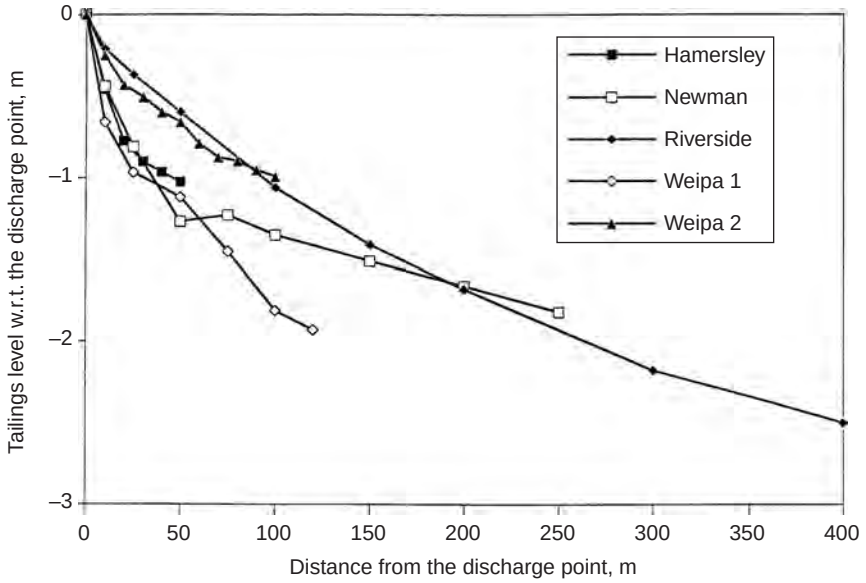


Figure 19.12 Measured beach profiles, fine grained high clay content tailings.

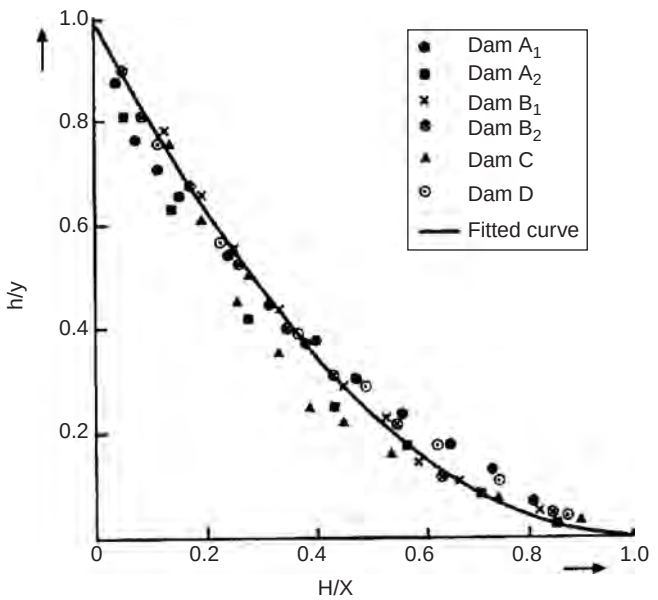


Figure 19.13 Dimensionless beach profile for platinum tailings in Figure 19.11 (Blight, 1988).

The master profile is different for each type of tailings. The particle size, specific gravity and solids content of the tailings all affect the profile. Blight (1988) indicates that the master profile applies, almost regardless of the length of the beach, and hence laboratory flume tests can be used to predict the behaviour of the dam.

Morgenstern and Kupper (1988) indicate that, in their experience, the laboratory flume tests did not show trends consistent with field behaviour.

The concept of a master profile is, however, well established and has been noted by others, e.g. Smith *et al.* (1986). Hence, one can predict later performance quite well from the initial behaviour of a tailings storage.

For planning purposes in a new project one will have to rely on flume tests and relationship to other measured profiles for similar tailings.

De Groot *et al.* (1988) point out that the observed slopes in dredged sands below water are much flatter than the predicted equilibrium slope (which might be expected to be steeper than slopes above water because of reduced gravitational forces below water). This is explained as due to flow slides where the upper part of a slope over-steepens as sedimentation occurs, resulting in slope instability and flow which produces flatter slopes. They note that observed slopes below water were similar to that in the beach above water. This is also the authors' usual observation for high clay content tailings although in an alumina red mud storage the slope was steeper below water.

19.4.2 Particle sorting

In conventionally thickened tailings, because the coarser particles contained in the tailings settle more rapidly than the finer particles, a gradation of particle size occurs on the beach, with coarser particles depositing near the discharge point. As discussed above particle sorting also occurs within a layer, with coarse, high specific gravity particles settling to the base of a layer. These can form a sand 'parting' even in high fines content tailings. Conversely, if the coarse particles are low specific gravity as occurring in coal washery tailings, they will 'float' to the surface of a layer.

The lateral variations in grading with distances from the discharge point can have an important effect on tailings permeability – with the coarser more permeable tailings deposited near the discharge spigots and the finer low permeability slimes deposited further away. This is used in the upstream method of tailings dam construction to control seepage pore pressures and, hence, to maintain stability. Blight and Bentel (1983) and Blight (1987, 1988) discuss this feature and conclude that the particle size distribution can be roughly predicted from:

$$A = e^{-BH/X} \tag{19.13}$$

where

$$A = \frac{D_{50}\ (\text{at distance H down the beach})}{D_{50}\ (\text{of the total tailings})} \tag{19.14}$$

B = characteristic of tailings and is a function of the discharge rate, and the specific gravity of the tailings;

X = length of beach as in Figure 19.11.

Abadjiev (1985) presents a similar formula which can be applied to not only the D_{50} size but to D_{90}, D_{60}, D_{10} etc. He also presents data from several tailings dams showing the lateral gradation.

The authors' experience is that the amount of sorting which occurs is dependent on the method of deposition. Spray bars and closely spaced spigots give very good sorting provided discharge rates are kept low. This is consistent with the observations of Blight and Abadjiev.

It should be noted that for tailings thickened to 50% to 60% solids content there is no sedimentation phase during the deposition, little or no bleeding of water/liquor and little or no particle sorting. Hence there will be more uniform particle size and deposited slope than for tailings thickened to only say 35%–40% solids content.

19.4.3 Permeability

The permeability of tailings is dependent on the particle size distribution and void ratio, so is therefore dependent on the distance from the discharge point, the method of deposition, e.g. Sub-aqueous or sub-aerial, and the depth in the storage.

The following procedure is therefore recommended for the estimation of permeability at any particular place:

(1) Predict the particle size at that place from the grading of the whole tailings and the sorting as predicted by the approaches discussed above in Section 19.4.2.
(2) Predict the void ratio from sedimentation, and the sorting as predicted by the approaches discussed below in Section 19.4.4.
(3) Prepare samples based on (1) and (2) above.
(4) Calculate permeability from either falling head or constant head tests, or by back calculation from consolidation tests, conducted on these samples.

For sand sized tailings the permeability can be approximately estimated from Hazen's formula, i.e.

$$k = D_{10}^2/100 \tag{19.15}$$

where k = permeability in m/sec; D_{10} = grain size in millimetres, for which 10% of the particles are finer.

Using this and the relationship between particle size and distance from the discharge point discussed in Section 19.4.2, Blight (1987) suggests that the permeability of the beached tailings will vary as:

$$K = ae^{-bH} \tag{19.16}$$

where 'a' and 'b' are dependent on the tailings; H is the distance down the beach.

As described in Winkler *et al.* (2014a) CPTu dissipation tests can be used to determine the permeability of tailings *in situ*. This has the advantage that the permeability is determined for the actual tailings deposition characteristics and stress conditions of the actual deposition of the tailings and, as these vary spatially with distance from the spigot discharge point, and with depth and effective stress.

19.4.4 Dry density

Prediction of the dry density of tailings in the storage is of prime importance, because the amount of tailings which can be stored in a given storage volume depends on the density achieved. This, in turn, is dependent on the type of tailings, method of deposition (subaqueous or sub-aerial), drainage conditions (e.g. if under-drains are provided or if the soil and rock underlying a tailings dam is of high permeability) degree of desiccation, distance from the discharge point and proximity to the water pond etc.

For tailings which are deposited under water there are two phases:

(a) Sedimentation
(b) Consolidation under self weight and the weight of tailings above

The *sedimentation phase* occurs when the tailings are first placed in the dam, and the particles settle, with a clear water interface forming on the surface of the tailings. In sub-aerial deposition this 'bleed' water will run off down the beach slope.

The *consolidation phase* follows and may occur concurrently with sedimentation. Prediction of consolidation of tailings requires a different approach from that for normal soils, because of the large strains involved. The traditional one-dimensional Terzaghi theory is not applicable and would result in overestimation of the time of consolidation and underestimating pore pressures. It is also necessary to use special slurry consolidometers, so tests can begin at the density of the tailings after sedimentation.

Schiffman and Carrier (1990) and Schiffman *et al.* (1988) give overviews of this topic, and reference other papers which can be used to obtain the detail of the testing and analysis techniques. They indicate that centrifuge testing can also be used for more accurate modelling, but advocate monitoring and analysis of the behaviour of the early phases of tailings deposition as the best means of predicting behaviour.

The reader is referred to these references rather than have the information repeated here. However, the following practical points are based on the authors' experience:

- It is essential to carry out testing with tailings and water which are representative of the actual tailings and water. It is also essential that the tailings have not dried out before carrying out testing, particularly if the tailings have significant clay content. Drying completely changes the sedimentation behaviour (usually makes the tailings settle more quickly and more densely) and affects the consolidation and permeability properties
- Sedimentation tests, where the tailings are mixed and placed in a 500 mm column and allowed to settle, give a good indication of the minimum settled density. Figure 19.14 gives the results of such tests for the tailings in Tables 19.2 and 19.3 in 500 mm columns. Field measurements indicate that the 500 mm columns slightly underestimate the field density of tailings.
- Field sampling of the very loose/soft upper layer of tailings is difficult. Ritcey (1989) refers to the use of piston samplers. Fell *et al.* (1992) show a simple device, which was developed by R. Hogg at the University of New South Wales has been used to obtain disturbed samples of tailings. This allows determination of water content, and hence dry density or void ratio, and give samples for particle size distribution testing.

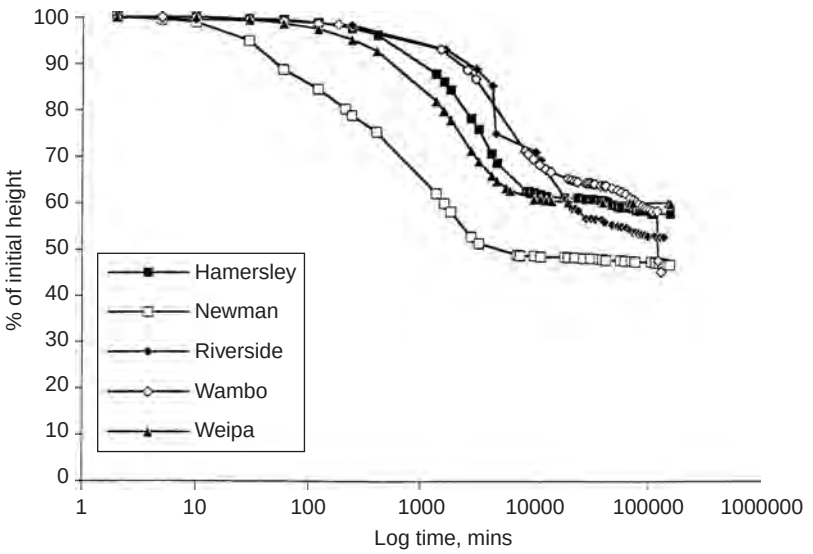


Figure 19.14 Settling tests on mine tailings.

19.4.5 The prediction of desiccation rates

If one is to rely on the increased dry density and strength which can be obtained from desiccation of the tailings, it is necessary to predict the rate of drying. The rate is dependent on:

- The properties of the tailings, in particular the settled water content, the permeability and the suction pressure which develops on drying.
- The climate – evaporation and rainfall.
- The deposition cycle – depth of and time between each cycle of deposition.

In Swarbrick (1992) and Swarbrick and Fell (1991) the results of a research program to develop a method for predicting desiccation rates are described. Based on laboratory and field drying experiments, it has been shown that desiccation occurs as:

(a) Settling until the rate of water release equals the potential evaporation.
(b) Stage 1 drying, which occurs at a linear rate with time, generally at the same rate as from a free water surface.
(c) Stage 2 drying, which occurs at a decreasing rate. This decreasing rate has been shown to satisfy the sorptivity equation, i.e.

$$E_{cum} = b\sqrt{t} \tag{19.17}$$

where E_{cum} = cumulative evaporation after the linear stage = time after the linear stage (days); b = sorptivity coefficient (mm days$^{-0.5}$).



Figure 19.15 Rotating screw device for breaking up the salt layer on top of alumina red mud tailings (photograph courtesy of Geographe Earthmoving).

Swarbrick (1992) Swarbrick and Fell (1991) give details of the method for carrying out laboratory drying tests on samples 600 mm × 600 mm × 300 mm to provide the data required for use of the method.

It is very important to note that the drying of tailings is dramatically affected by the presence of salts in the water/liquor. This is a particular problem in alumina red mud where the salts are NaOH and $NaCO_3$, but is also a problem in other tailings such as from coal washeries where natural groundwater is highly saline, or if saline water is used for processing or neutralizing the tailings. The salt concentrates on the surface, and the osmotic suction slows and eventually stops evaporation from the tailings surface. To keep desiccation continuing it is necessary to break up the salt on the surface using special equipment such as that shown in Figure 19.15 or using swamp dozers once the tailings are strong enough for them to work.

19.4.6 Drained and undrained shear strength

19.4.6.1 *Drained shear strength*

The effective strength parameters c', ϕ' for tailings, can be obtained from triaxial or direct shear tests on samples of the tailings. It is particularly important that sandy tailings are tested at the correct relative density and stress range, because of the strong dependence of ϕ' on the relative density and the curved nature of the Mohr envelope.

Table 19.7 shows some typical values of effective friction angle. It will be noted that most are relatively high, reflecting the absence of any structure and the grinding of rock to angular particles. Unless the tailings become cemented on deposition, e.g. by gypsum, the effective cohesion is $c' = 0$.

As discussed below, it will in many cases be wrong to rely on the drained shear strength of the tailings because if they are loose and saturated, they will be contractive

Table 19.7 Typical values of effective friction angle ϕ' of tailings (adapted from Vick, 1983).

Material	ϕ' *(degrees)*	*Effective stress range kPa*(1)
Copper		
Sands	34	750
	33–37	625
Slimes	33–37	625
Taconite		
Sands	34.5–36.5	–
Slimes	33.5–35	–
Lead-zinc-silver		
Sands	33.5–35	–
Slimes	30–36	–
Gold slimes	28–40.5	900
Fine coal refuse	22–39	270
	22–35	1100
Bauxite slimes	42	175
Gypsum tailings	32 ($c' = 22$ kPa)	450

Note: (1) From zero to these maximum values.

on shearing and develop positive pore pressures. The undrained strength is a more reliable way of assessing their strength. The beginning of contractive behaviour leads to a sudden increase in pore pressures at the collapse surface, and is a lower strength envelope than the drained shear strength (see Figure 6.10).

19.4.6.2 *Undrained shear strength*

(a) *Plastic tailings*

The undrained shear strength of the 'slimes' or clay-silt sized part of the tailings can be important both from the view of overall slope stability in upstream construction and the strength of the surface of the tailings for rehabilitation.

The strength can be determined by appropriate triaxial testing, e.g. using the SHANSEP method (Ladd and Foott, 1974), but must be related to the field situation by a knowledge of the degree of over-consolidation of the tailings. This can be done by taking undisturbed samples of the tailings and carrying out oedometer consolidation tests. However obtaining undisturbed samples of very soft tailings is difficult.

Alternatively, if consolidated undrained triaxial tests with pore pressure measurement are carried out on undisturbed samples of the tailings, the pore pressures developed during shearing can be measured and the Skempton A_f at failure determined. Then the undrained strength can be determined using Equation 6.17, Chapter 6.

The undrained strength of fine silt and clay tailings *in situ* may be measured using a vane shear and CPT or CPTu. However the results are often very scattered because of the varying degrees of over-consolidation within layers which have been partly desiccated and the presence of sandy layers which lead to overestimation of undrained strength. Figure 19.16 gives some results from the Newman tailings, which show the wide scatter and apparent lack of relationship to water content.

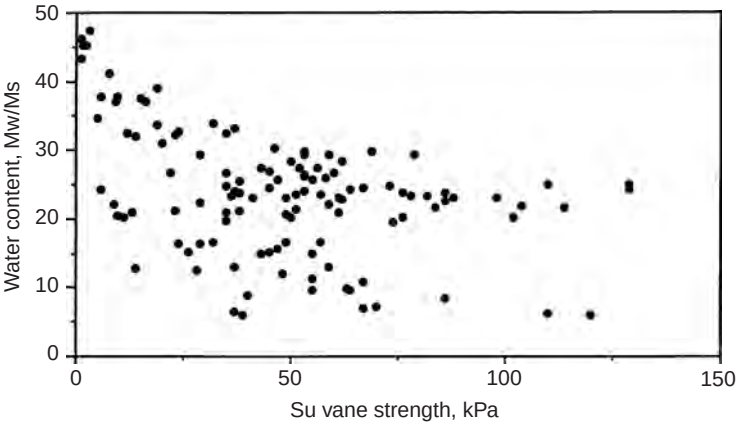


Figure 19.16 Vane shear test results – Newman tailings.

Where tailings are consistently fine grained, e.g. well away from the discharge point and not affected by desiccation other than in a single drying phase, reasonably constant strengths can be obtained by a vane shear.

(b) *Non-plastic tailings*

Loose non-plastic tailings will also be contractive in shear so the strength is overestimated using effective strengths and the pore pressures which exist before shearing. The undrained shear strength should be estimated using the method described in Section 6.1.7.1. Laboratory triaxial tests on samples sedimented to the field void ratio will assist in determining the input parameters and directly estimating undrained strength at the consolidation stresses.

As discussed in Winkler *et al.* (2014a) it is important to determine the *in situ* stresses from measured pore pressures within the tailings. These are affected by underdrainage and perched water tables and will vary considerably from near the discharge point towards the decant pond.

19.5 METHODS OF CONSTRUCTION OF TAILINGS DAMS

19.5.1 General

In most cases the basic requirements for a tailings disposal area are to store the tailings in such a way that they remain stable (i.e. there are no slope stability problems) and do not impact excessively on the environment by water pollution and/or wind and water erosion. Only in some cases is it necessary, or desirable, to store water in the tailings disposal area.

This should be kept in mind when designing a tailings embankment and one should not always feel it necessary to construct 'a dam' to store the tailings.

Many of the principles and practices developed in conventional water dam engineering are applicable to tailings, but it should not be assumed that all water dam technology or philosophy is applicable.

In many cases it is better to consider tailings embankments as landfills, or dumps of fine grained soil which have to be engineered to:

- Optimise the amount of tailings which can be stored in a particular area.
- Control piezometric conditions to ensure adequate strength and stability.
- Control the environmental impact.

Australian practice has in the past been (and to a lesser extent still is) biased towards constructing conventional dams to store tailings. This approach also applies in some other countries. This is possibly due to:

- Many designers have a background in water dam engineering.
- Regulatory authorities require such an approach (largely because of their background in water dam engineering).
- A relatively wet climate in many mining areas and many mines with very fine grained and/or oxidised tailings which are not readily utilized in 'upstream' construction methods.

There is recognition amongst many practitioners that this philosophy is costly and unnecessary and alternative methods are being used.

The following discussion is intended to give an overview of the various alternative methods and their limitations, with a view to encouraging their consideration and adoption where appropriate.

19.5.2 Construction using tailings

There are three main methods for constructing tailings embankments using the tailings as a major construction material:

- Upstream method
- Downstream method
- Centreline method.

These are described in some detail in ICOLD (1982, 1996) and Vick (1983).

19.5.2.1 *Upstream method*

The important features of this approach (Figure 19.17) are:

- The starter dam is essentially a containing embankment and a support for the tailings discharge line, rather than a dam in itself. The starter dam is best constructed of permeable rockfill (e.g. mine waste) to allow drainage of seepage water and to control erosion. However, it may be constructed of relatively impermeable rockfill, earthfill or even dried tailings pushed up by bulldozers from the tailings beach.
- Tailings discharge must be controlled by e.g. spigotting to ensure that the coarser sandy tailings are deposited near the starter dam. This is essential to control seepage pressures as outlined below.
- The water pond (decant pond) must be kept well away from the edge of the storage. If allowed to come close to the edge, the piezometric pressures will be high and

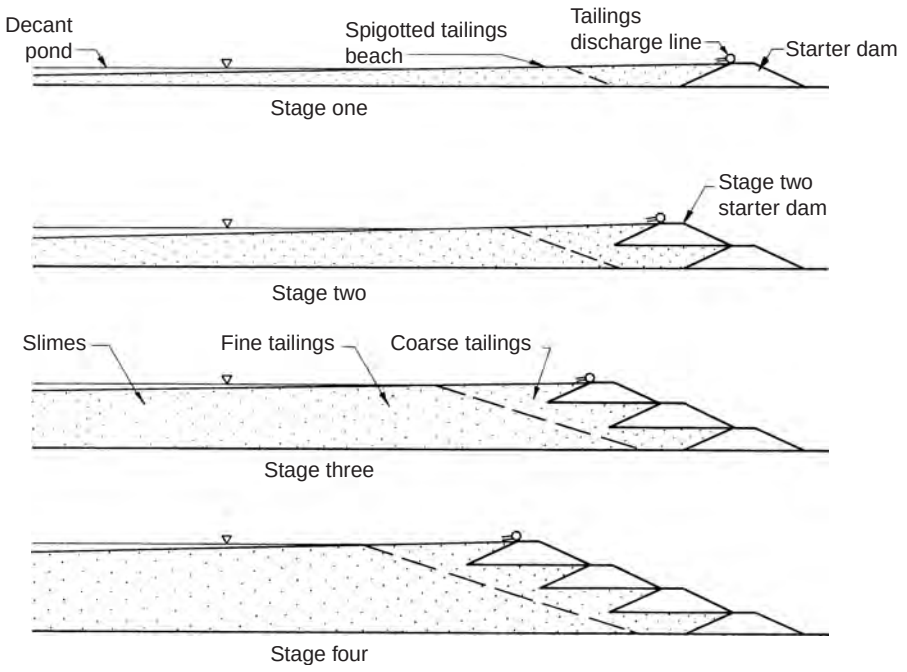


Figure 19.17 Construction of a tailings embankment using the upstream method.

slope instability may result, or internal erosion and piping of the starter dam or of the tailings into the starter dam may occur.

– The coarse tailings are likely to be at a low to medium relative density and, if saturated and subjected to earthquake, may liquefy, leading to slope failure with a subsequent flow failure of the tailings. Because of this the method is seldom used in seismic risk areas. It should be noted that because of layering the tailings are likely to have perched water tables, so saturation is almost inevitable.

The method is best suited to hard rock tailings, i.e. silt-sand tailings which are readily spigotted to classify into a sandy beach, but can be applied also to high clay content tailings.

Stability will also be dependent on allowing the finer tailings ('slimes') to desiccate and develop a significant undrained strength.

Figure 19.18 shows use of the upstream method for high clay size content bauxite tailings at Weipa (Weipa 1 tailings in Figure 19.5).

19.5.2.2 *Downstream method*

Figures 19.19 and 19.20 show two versions of the downstream method of tailings embankment construction.

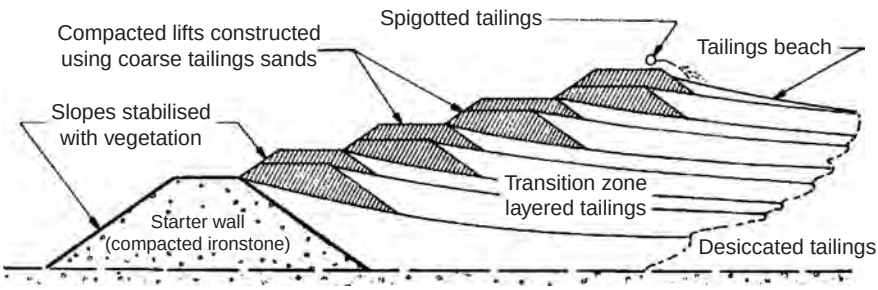


Figure 19.18 Section of bauxite tailings embankment at Weipa showing upstream construction (Minns, 1988).

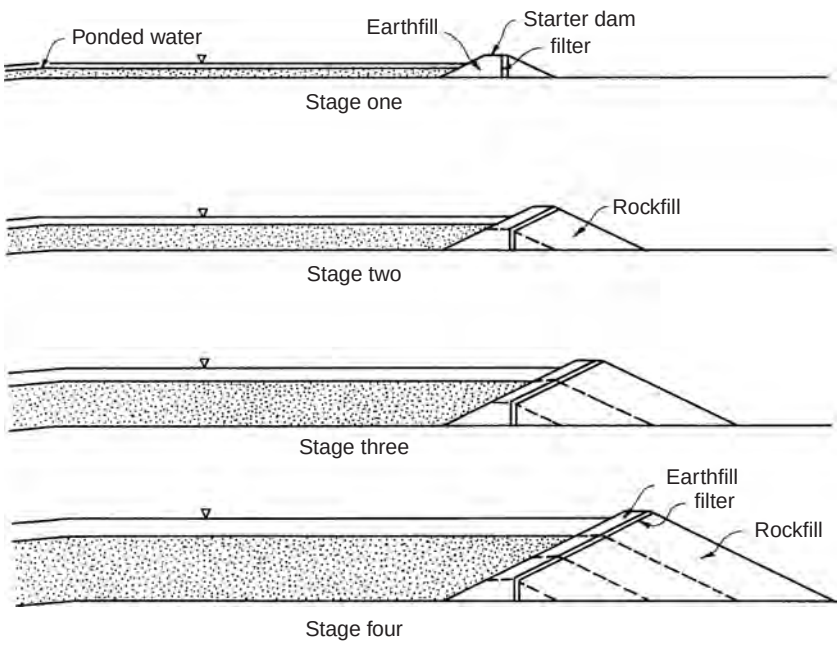


Figure 19.19 Tailings embankment construction using the downstream method and zoned earth and rockfill construction.

The important features of this approach are:

– The embankment is constructed of selected material. In the case of Figure 19.19, a water storage dam type cross section has been used, with an upstream impervious zone and internal drainage control. This allows water to be stored adjacent to the embankment. In Figure 19.20 the embankment is constructed from the coarse underflow part of cycloned tailings. These may be compacted by tracking with small bulldozers as described in Section 19.3.2 and in Davidson and Winkler (2014). Avoidance of internal erosion and piping is dependent on the

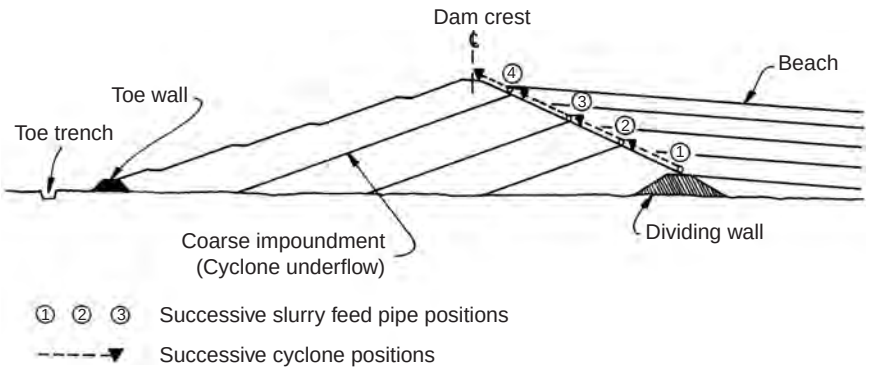


Figure 19.20 Tailings embankment construction using the downstream method and cycloned tailings (ICOLD, 1982b).

water pond being kept well away from the embankment to control seepage gradients in non-plastic tailings and to keep water from entering cracks in plastic and high silt content tailings which may hold a crack and therefore be susceptible to concentrated leak erosion.

– The embankment is best constructed, at least partly, of permeable material to allow control of piezometric pressures.
– It is desirable, but not necessary, to place the tailings in a controlled manner. For Figure 19.19 type construction no control is necessary (from a stability viewpoint). For Figure 19.20 type construction it would be important to spigot or cyclone the tailings to form a relatively sandy beach adjacent the downstream zones to control piezometric pressures.

If rockfill rather than tailings underflow is used for the downstream zones in Figure 19.20, a graded filter zone will be required between the tailings and the rockfill to prevent erosion of the tailings into the rockfill which could lead to piping failure.

A combination of downstream and upstream methods may be used. An example is shown in Figure 19.21. There are a number of examples of this that the authors are aware of. The approach has been used because of the high costs of incremental downstream construction as the dam becomes high.

19.5.2.3 *Centreline method*

Figures 19.22 and 19.23 show two versions of construction by the centreline method.

The important features of this approach are:

– The embankment is partly constructed of selected material. In Figure 19.22 a zoned water storage type section has been used for the downstream part of the embankment, with the upstream part constructed on spigotted tailings. In Figure 19.23 cycloned tailings have been used, with the cyclone underflow discharged on the downstream side and the overflow finer portion on the upstream

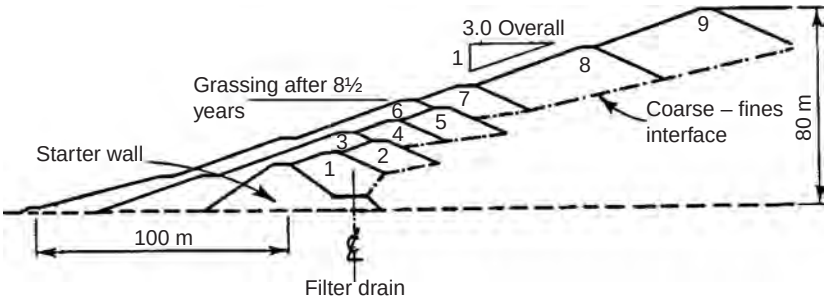


Figure 19.21 Combined upstream and downstream construction (Lyell and Prakke, 1988).

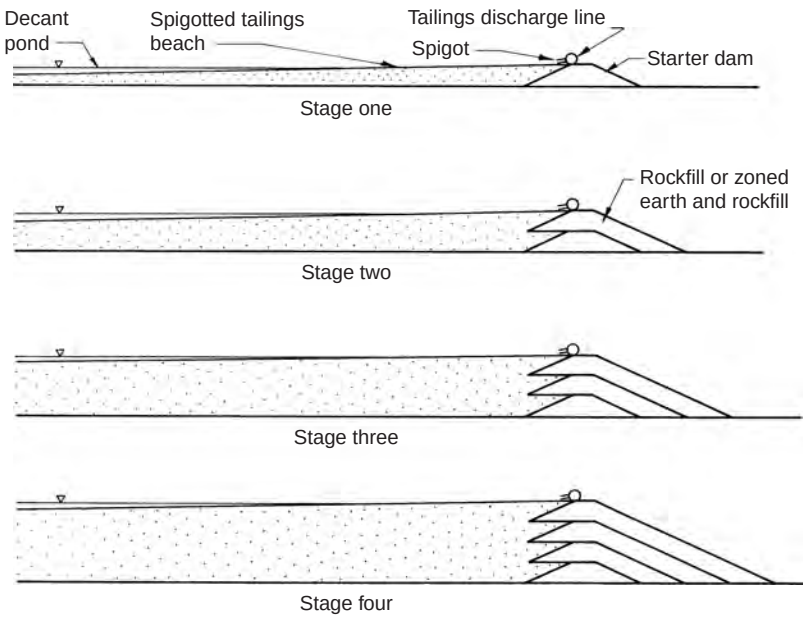


Figure 19.22 Construction of tailings embankment using the centreline method.

side. The embankment must be constructed, at least in part, of permeable material to allow control of piezometric pressure.

- The water pond must be kept away from the edge of the embankment to prevent build up of excessive piezometric pressures and to control potential internal erosion and piping.
- It is essential that tailings are placed in a controlled manner, i.e. spigotting for Figure 19.22, cycloning for Figure 19.23. For Figure 19.23 the tailings underflow would need to be compacted to ensure that it does not liquefy under earthquake.
- Weber (2014) and Davidson and Winkler (2014) give examples of centreline construction using rockfill and tailings for construction.

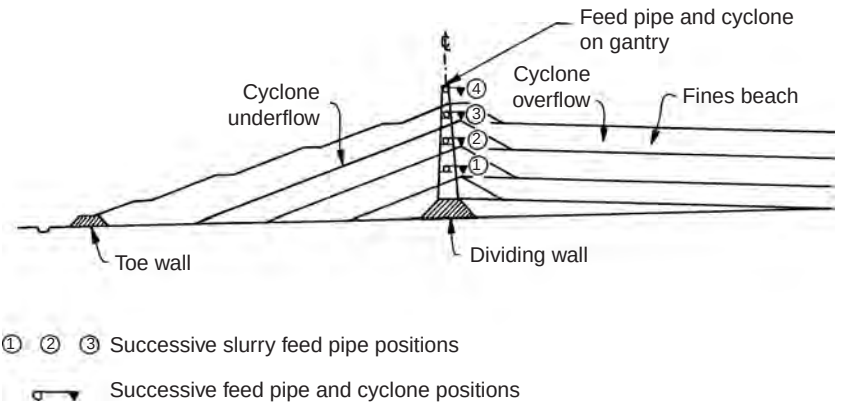


Figure 19.23 Construction of tailings embankment using the centreline method and cycloned tailings (adapted from ICOLD, 1982b).

19.5.3 Construction using conventional water dams

Conventional earthfill, earth and rockfill and concrete faced rockfill dams are often used to store tailings.

Some factors which may be considered as variations on conventional water dam engineering are:

- The tailings seepage water often has a high salt content, which suppresses the likelihood of dispersion of clays so marginally coarser filters may be used for dispersive soils. Leakage of water from the dam may not be critical if the seepage water has no contaminants, e.g. in some iron ore and bauxite washing operations, allowing the use of higher permeability core material such as the coarse fraction of tailings and limited foundation grouting. However if the seepage water is contaminated, seepage collection downstream may be necessary or liners and under-drains provided.
- There is often a large quantity of waste rock available from the mining operation which can be used for construction at a low cost. In this situation it may be possible to have placement and compaction of the rockfill being done by mining equipment. If a well controlled earthfill zone with filters is required (i.e. sloping upstream core construction), this would usually be built by earthworks contractors after the rockfill, with well controlled construction. The rockfill should be compacted to limit settlements. The rockfill should be checked for the presence of pyrite which can lead to acid mine drainage problems.
- Erosion of the upstream slope is seldom a problem for tailings dams. At most, waste rock should be dumped on the upstream face. Steeper upstream slopes can be used if the slope is supported by tailings and not subject to drawdown
- Staged construction is usually required, favouring sloping upstream core type cross sections.

Figure 19.24 shows an example of staged construction which minimises the volume of fill in each stage.

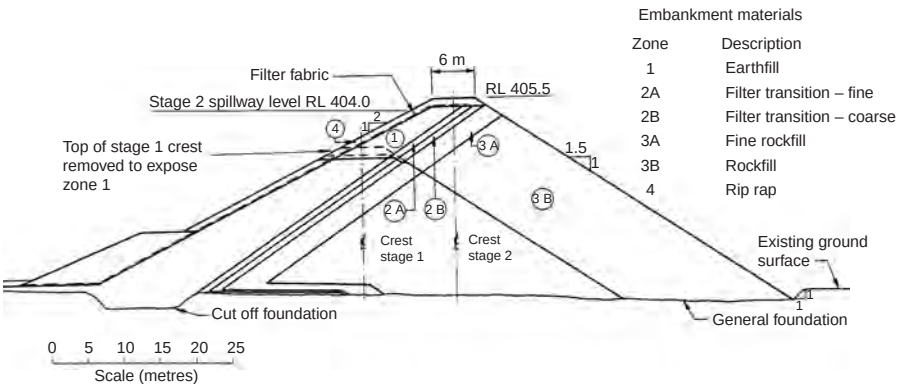


Figure 19.24 Staged construction – proposed Ben Lomond tailings dam (Coffey and Partners, 1982).

19.5.4 Selection of embankment construction method

The method to be used will depend on the particular circumstances at the mine, e.g. type of tailings, production rate, climate, local topography, seismicity, availability of waste rock, regulatory authority requirements etc.

All methods allow staged construction of the embankment. This is the significant difference between water storage dams and tailings embankments and allows minimisation of up-front capital works and improvement of overall economies.

Table 19.8, which is adapted from Vick (1983), compares the different methods of embankment construction. One important factor in selection of embankment type is the degree of seepage control achievable. This is vital, because it determines the stability of the embankment and the steepness of the downstream slope. Figure 19.25 shows that downstream construction allows best control, followed by the centreline method, and upstream construction. Figure 19.26 shows that, by providing internal drainage layers control of seepage can be engineered. However this is only achieved at a cost penalty. Blanket drains may not be totally effective if the tailings are layered (as often occurs) because the water table perches on low permeability layers of tailing.

19.5.5 Control of seepage by tailings placement, blanket drains and under-drains

19.5.5.1 *Tailings placement*

Vick (1983) and Blight (1987, 1988) discuss the benefits of sorting the tailings particles in the beach zone. This sorting can lead to a gradation of permeability from high near the discharge point decreasing towards the pond and hence a lower phreatic surface. Figure 19.27 shows this effect schematically and the effect of having the water pond near the embankment crest and the underdrainage effect of a high permeability soil or of a rock foundation.

Table 19.8 Comparison of tailings embankment types (adapted from Vick, 1983).

Embankment type	*Mill tailings requirements*	*Discharge requirements*	*Water storage suitability*	*Seismic resistance*	*Raising rate restrictions*	*Embankment fill requirements*	*Relative embankment cost*
Water retention	Suitable for any type of tailings	Any discharge procedure suitable	Good	Good	Entire embankment constructed initially or in stages	Earthfill, rockfill, (mine waste, filters)	High
Upstream	At least 40–60% sand in whole tailings. Low discharge solids content desirable to promote grain-size segregation	Peripheral spigotted discharge and well controlled beach necessary	Not suitable for significant water storage	Potentially poor in high seismic areas Unless specifically designed for seismic loads	Less than 5–10 m/yr most desirable	Natural soil, sand tailings, or mine waste	Low
Downstream	Suitable for any type of tailings	Varies according to design details	Good	Good	None. Often built in stages if rockfill is used	Sand tailings or mine waste if production rates are sufficient, or earthfill, rockfill	High
Centreline	Sands or low-plasticity slimes	Peripheral spigotted discharge of at least nominal beach necessary	Not recommended for permanent storage. Temporary flood storage acceptable with proper design details	Acceptable	Height restrictions for individual raises may apply	Sand tailings or mine waste if production rates are sufficient, or earthfill, rockfill	Moderate

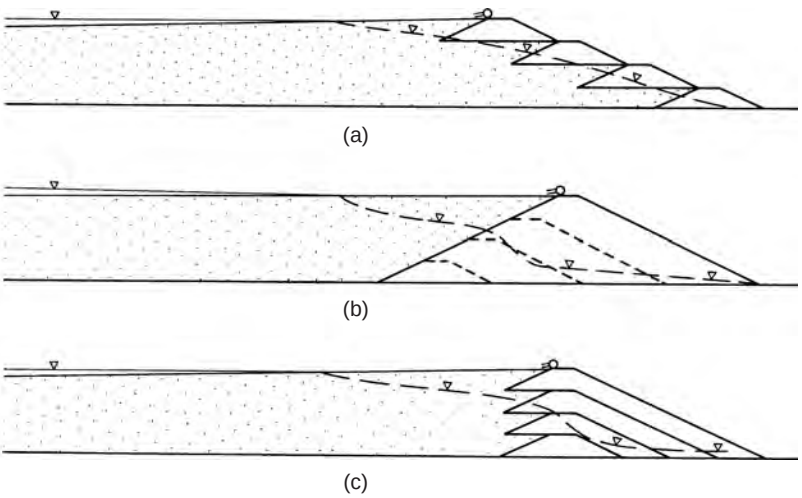


Figure 19.25 Internal seepage: (a) Upstream embankment; (b) Downstream embankment; (c) Centre-line embankment.

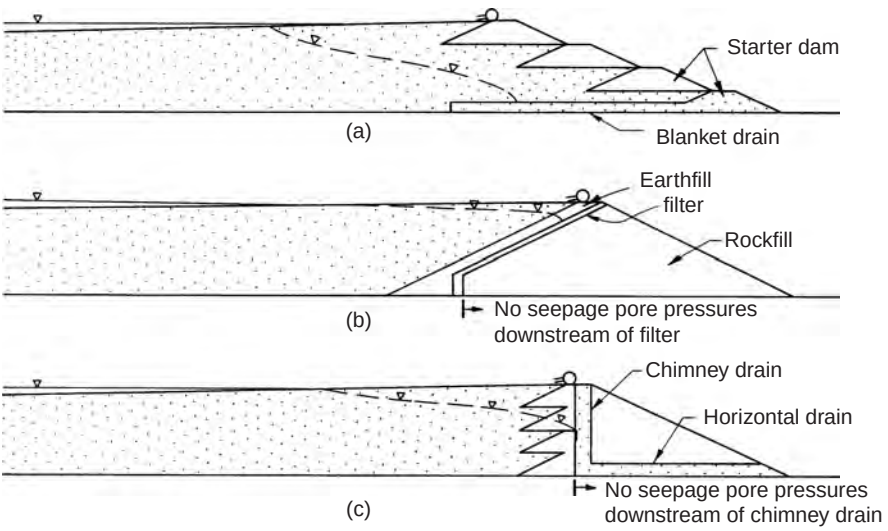


Figure 19.26 Use of internal drainage zones in embankments: (a) Upstream embankment using starter dam with upstream blanket drain; (b) Downstream embankment; (c) Centreline embankment with vertical chimney drain.

Stauffer and Obermeyer (1988) discuss a case where piezometric pressures were lower than might otherwise have been expected, because of the underdrainage effect of a relatively high permeability foundation.

In many designs, drainage zones will be incorporated into the design, within the embankment, as shown in Figure 19.28.

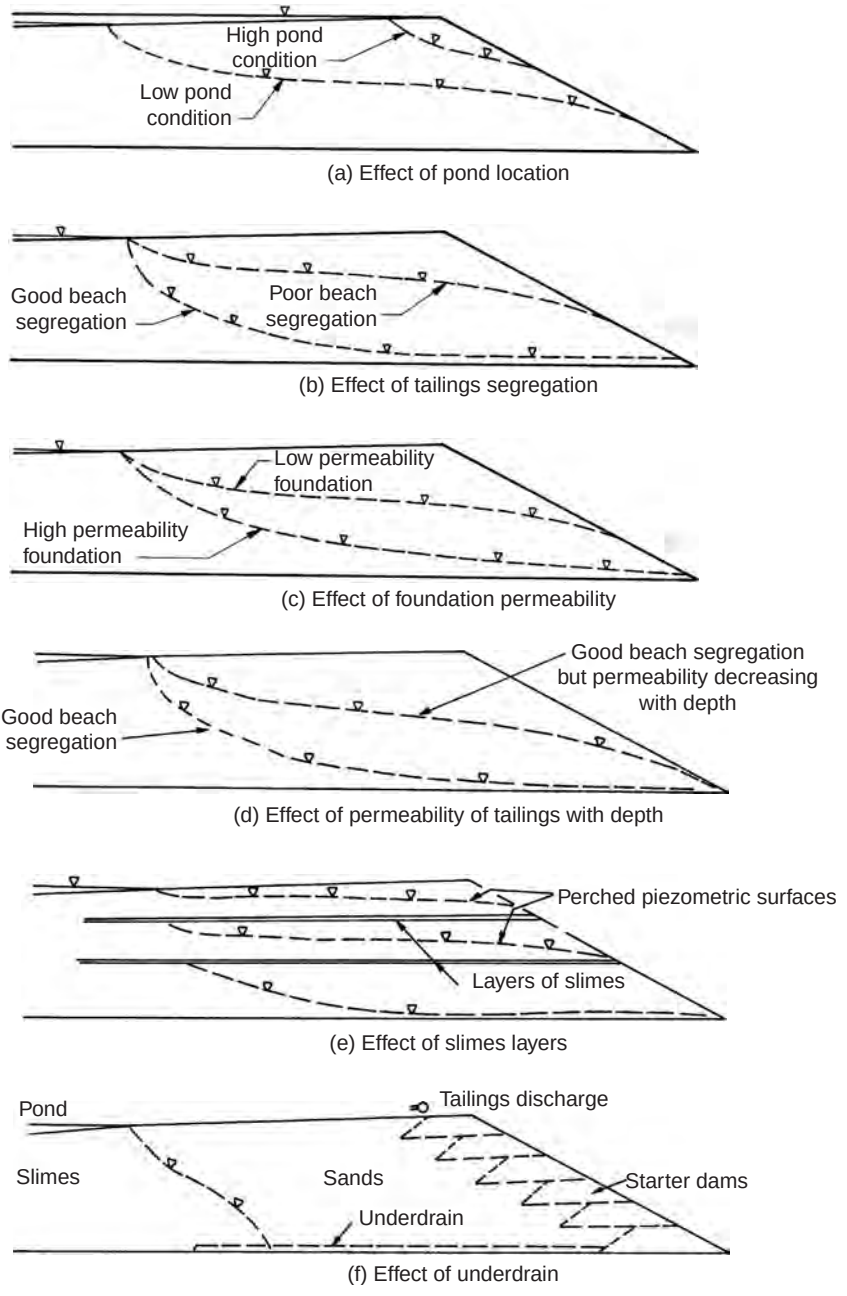


Figure 19.27 Factors influencing phreatic surface location for upstream embankments: (a) Effect of pond water location; (b) Effect of beach grain-size segregation and lateral permeability variation; (c) Effect of foundation permeability; (d) Effect of decreasing tailings permeability with depth; (e) Effect of slimes layers; (f) Effect of under-drains if there are no layers of slimes. Starter dams not shown in (a) to (e).

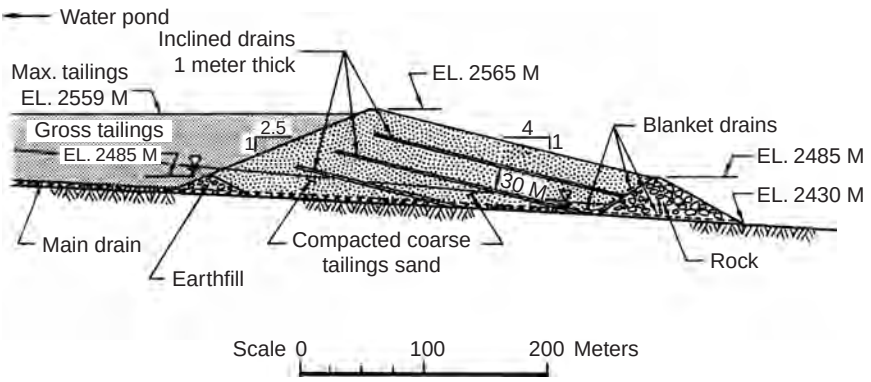


Figure 19.28 Embankment drainage in Perez Caldera No. 2 tailings dam (Griffin, 1990).

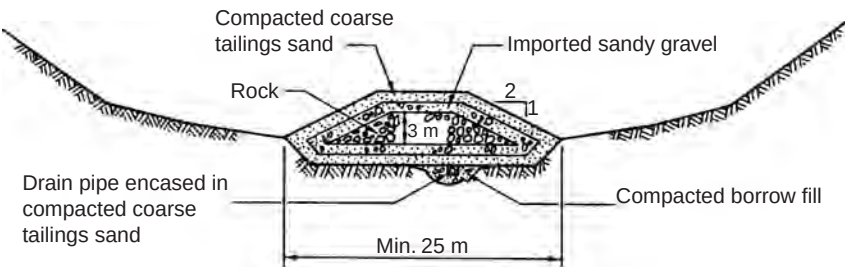


Figure 19.29 Perez Caldera No. 2 tailings dam under-drains (Griffin 1990).

19.5.5.2 *Drainage blankets and under-drains*

Drainage blankets as shown in Figure 19.26(a) and Figure 19.30 and under-drains as shown in Figures 19.28 and 19.29, are sometimes used to drawdown the phreatic surface and to collect contaminated seepage into a seepage collection system.

These drains have in the past often been built with mine waste, tailings or borrow pit run materials, without due regard for filter design between the tailings and the drain or sufficient discharge capacity and they have not functioned as desired. In any case, where the tailings are layered, with slimes or finer tailings between the sandy tailings, the water will perch on these low permeability strata and the drains will not be very effective.

The other point to remember, is that the foundation is often more permeable than the tailings, so the seepage may be vertically downwards without the under-drains, in which case they provide no benefit in reducing pore pressures.

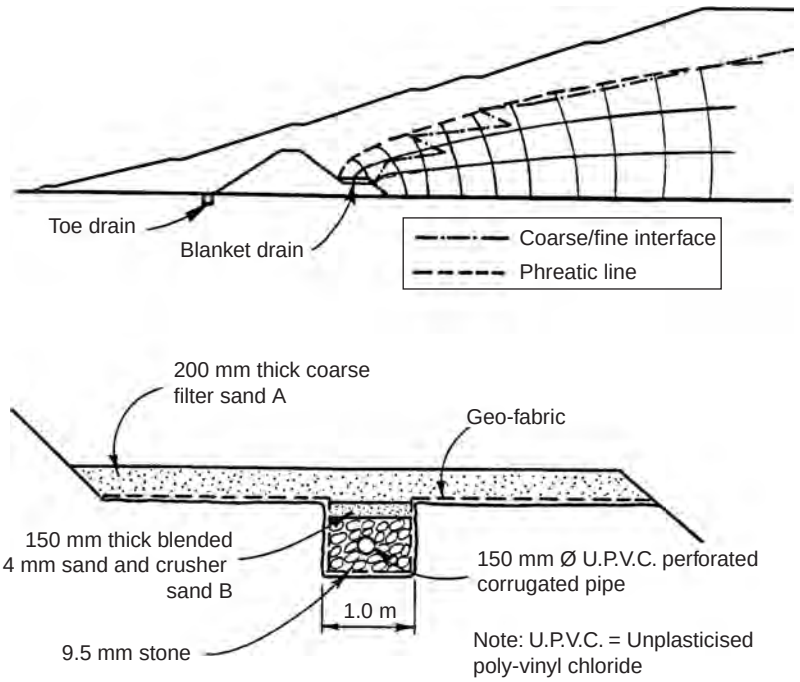


Figure 19.30 Blanket drain (for dam in Figure 19.22, Lyell and Prakke 1988).

19.5.6 Some factors affecting the potential for internal erosion and piping of tailings dams

The following summarizes some factors relating to the potential for internal erosion and piping of tailings dams:

(a) *Concentrated leak erosion*

Is a potential failure mode where conventional embankment dams are used to store tailings but only above the top of the tailings, or where the tailings are cracked by desiccation or differential settlement and can hold a crack in which concentrated leak erosion can initiate.

Below the top of the tailings the seepage gradient is reduced by the tailings and the tailings may act to give upstream flow limitation.

(b) *Backward erosion piping*

Is possible in non-plastic tailings provided there is a plastic layer or partially saturated high silt content layer which can provide the roof which is essential for backward erosion piping to occur.

(c) *Suffusion*

Most non-plastic tailings are too fine and uniformly graded to be susceptible to suffusion. Plastic tailings are not susceptible to suffusion.

(d) *Contact erosion*

Contact erosion is possible for all tailings in contact with open joints or other defects in the foundation.

For all of these internal erosion mechanisms, there would need to be an unfiltered exit for the tailings to be eroded for internal erosion to occur.

19.5.7 Some factors to consider for seismic design of tailings dams

19.5.7.1 *Conventional dams and downstream construction*

The seismic design of such dams to contain tailings is essentially as for water dams. One detail issue to consider is that if there are for example spillway walls against which tailings are deposited the load on the walls will be potentially large if the tailings liquefy. In effect they become close to water loads but with the water having the density of the saturated tailings.

19.5.7.2 *Upstream construction*

Upstream construction tailings dams are particularly susceptible to seismic loading because the tailings may liquefy. Historically there have been a number of failures, most well known being the El Cobre tailings dam in Chile which collapsed in 1965 as a result of liquefaction. The tailings flowed 12 km in minutes and killed more than 200 people. As a result upstream construction was banned in Chile and remains so.

The methods described in Chapter 12 are generally applicable to the design of these structures but there are some matters of detail which should be considered. These include:

Identification of tailings which are potentially susceptible to liquefaction

Reid (2012) warns that whether tailings are potentially liquefiable or not should be done using the Seed *et al.* (2003) and/or Bray and Sianco (2006) methods, not the Robertson and Wride (1998) or other methods which rely on $Ic > 2.6$ soils not being liquefiable. He lists the examples below to support this.

- Shuttle and Jefferies (2004), Ic 2.5–3.25, 30-50% fines, yet liquefiable.
- Shuttle and Cunning (2007), $Ic > 3.25$, 65% fines, yet liquefiable
- Anderson and Eldridge (2010), $Ic > 3.0$, 70–100% fines, yet liquefiable

In a recent seismic safety assessment of an important hydraulic fill dam for which the first author was an advisor the consultant based in USA, and its reviewer, one of the leaders in liquefaction research and practise, recommended using $Ic > 3.0$ as the limit of potentially liquefiable soil, which is consistent with what is discussed above. Winkler *et al.* (2014) describe a method for determining whether tailings will be contractive and hence potentially susceptible to flow liquefaction based on detailed analysis of CPTu tests carried out in the deposited tailings. This uses the dynamic pore pressure response as shown in Figure 19.31. Where the dynamic pore pressures are less that the hydrostatic pressures this indicated dilative behaviour (Figure 19.31a) and if positive, contractive behaviour (Figure 19.31b).

Winkler *et al.* (2014b) used the Been *et al.* (1986) state parameter concept to define the boundary between contractive and dilative behaviour at the $\psi = -0.05$ line on the normalised cone resistance versus normalised friction ratio plots as shown in

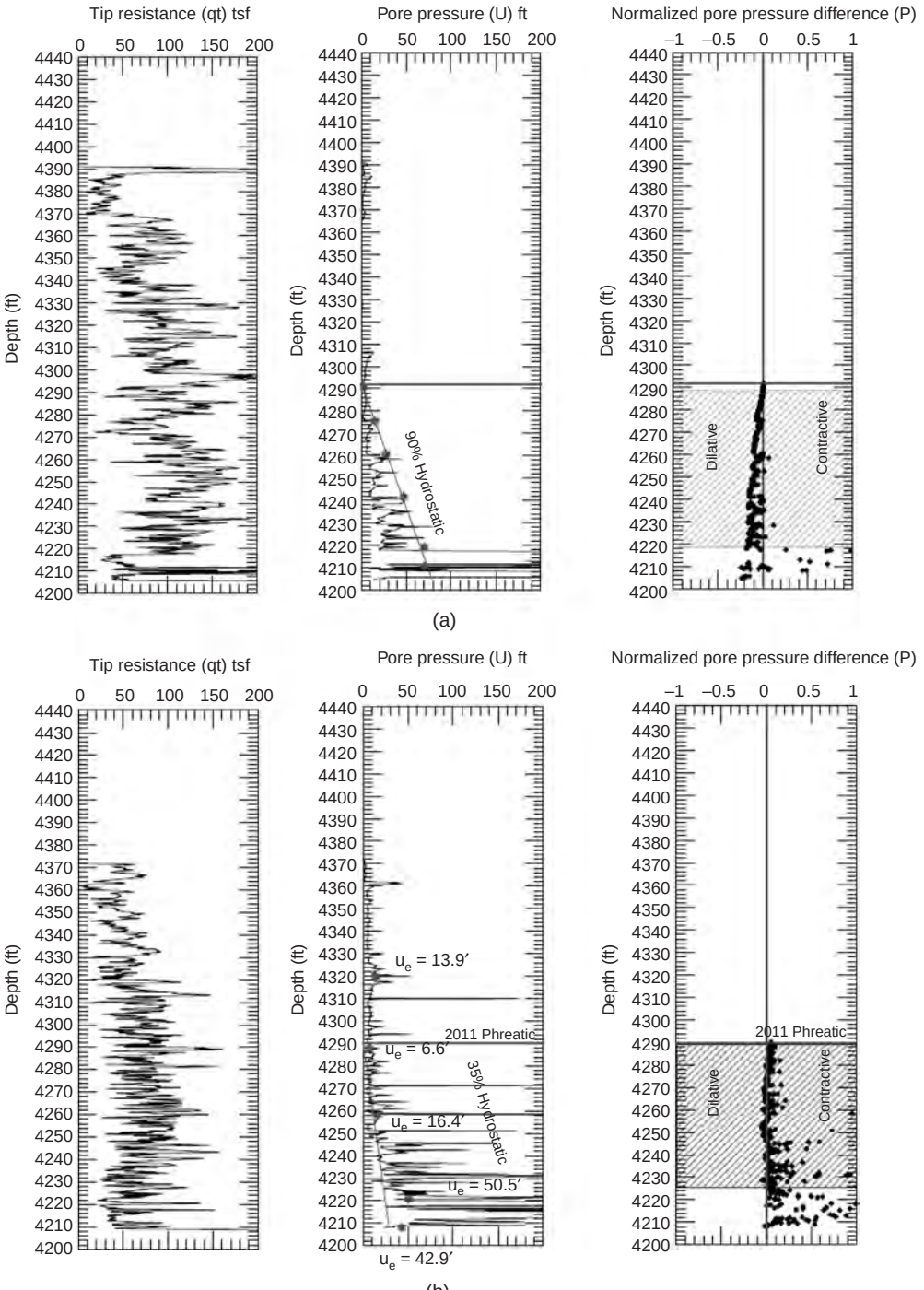


Figure 19.31 CPTU tests in tailings showing: (a) Dilative behaviour, (b) Contractive behaviour (Winkler *et al.*, 2014b).

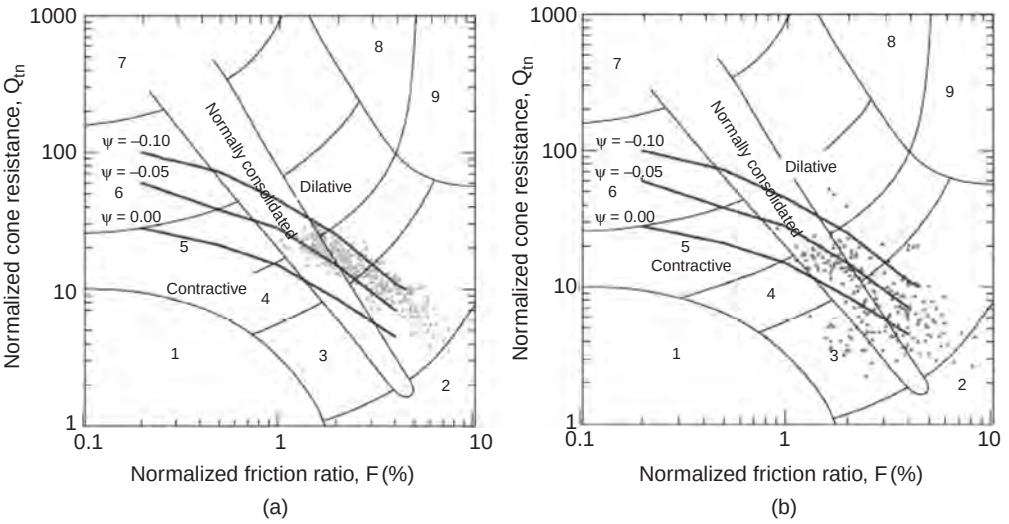


Figure 19.32 Normalised cone resistance versus normalised friction ratio for: (a) Typical dilative behaviour, (b) Typical contractive behaviour (Winkler *et al.*, 2014b).

Figure 19.32. They suggest this can be used as an additional screening tool along with the traditional CSR/CRR factor of safety.

Will under-drains result in unsaturated and hence non-liquefiable tailings?

The authors caution against assuming that because a tailings dam has an under-drain system the tailings above the under-drains will be partially saturated and hence non-liquefiable. Winkler *et al.* (2014a) give an example of where this has been investigated with some tailings unsaturated and others further from the discharge point more saturated.

If the tailing are stratified either on a large scale because of variations in tailings properties, or within any one layer because of the sedimentation effects described in Section 19.2.3.4, there will be perched water tables with saturated strata even though pore pressures may be very low. In any case under-drain systems are often rendered ineffective with time because of poor detailing and/or unavoidable chemical clogging. In these cases it may be unwise to rely on the tailings not being saturated when assessing liquefaction potential.

Use of simplified methods for assessing whether tailings will liquefy

For existing tailings dams the potential for liquefaction is best carried out using CPTu backed up by drilling, sampling and SPTs of representative strata. Idriss and Boulanger (2008) type methods are applicable to tailings although the database upon which the methods are determined have few tailings cases. The qualification of using the simplified methods for sloping ground as in a tailings dam, and high stresses for high dams apply equally to tailings dams as water storage dams.

Winkler *et al.* (2014a) point out that it is critical to determine the actual pore pressures in the tailings when carrying out such analyses.

Liquefied strength

The liquefied strength of tailings can be estimated by the methods described in Section 12.5.6 based on CPTu's and SPT. There appears to be more willingness to rely upon the strengths obtained in cyclic laboratory tests on tailings than for naturally occurring soils. These seem often to give larger strengths than the methods based on CPTu and SPT. The methods based on CPTu and SPT allow for void and pore pressure redistribution, and particle intermixing which are not replicated in the laboratory. If the tailings deposits are very uniform, with a lack of layering and permeability contrasts there may be a reasonable case for using strengths based on well executed laboratory methods. For important tailings dams expert advice should be obtained if strengths greater than those from the CPTu and SPT methods are being relied upon.

Use of critical state based methods

There are some who advocate the use of critical state based methods for assessing liquefaction potential and liquefied strengths based on the state parameter ψ which is the difference between the *in situ* void ratio and the critical state void ratio. These methods are described in Been and Jefferies (1985), Jefferies and Been (2006), Jefferies and Shuttle (2011). The authors have no experience in such methods so cannot comment in any detail. Reid (2012) indicates that the methods are limited by the scatter in relationship between cyclic resistance ratio CRR and ψ (CRR is dependent on more than just density) and ψ can only be estimated *in situ* with about $+/-0.05$ accuracy, and this potential error is transferred on to CRR estimate. Reid (2012) also points out that any tailings deposit has a range of critical state soils within it which has to be accommodated in the analysis.

Estimation of deformations

It should be noted that the simplified methods for estimating deformations under seismic loading described in Sections 12.6.5 to 12.6.7 do not apply to upstream or centreline construction tailings dams, in particular when liquefaction occurs. It is necessary to use numerical methods as described in Section 12.6.8.

19.5.8 Storage layout

Some common storage layouts are shown in Figures 19.33 and 19.34.

Some features of these arrangements are:

Ring dike

- Suitable for flat terrain
- No runoff from external catchments
- Can be staged in plan
- Allows cycling of tailings disposal into separate cells.

Cross valley

- Should be sited at head of valley to limit external catchment
- Usually involves less embankment material than ring dike
- Can be staged either in height or by building multiple impoundments.

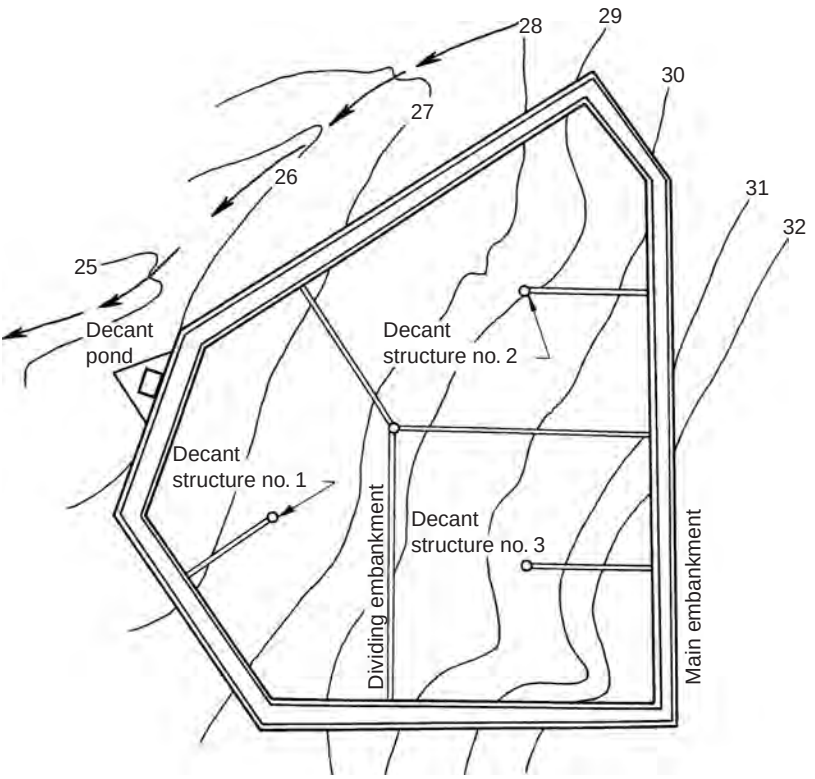


Figure 19.33 Ring dike or 'turkey's nest' configuration as used in Paddy's Flat tailings storage (adapted from Cooper, 1988).

Side hill

– Suitable only where hill slopes are relatively flat – say 10%. Otherwise embankment fill volumes become excessive.

In many cases it is desirable to limit the flow into the storage from external catchments using catch drains such as those shown in Figure 19.35.

19.5.9 Other disposal methods

There are other less commonly used methods of tailings disposal. These include:

19.5.9.1 *Thickened discharge or Robinsky method*

In this method tailings are thickened to a higher solids content than would normally be used, i.e. around 60% solids content compared to 30% to 40% for normal tailings operation. At this solids content the tailings can be deposited in a cone shaped deposit as shown in Figure 19.36.

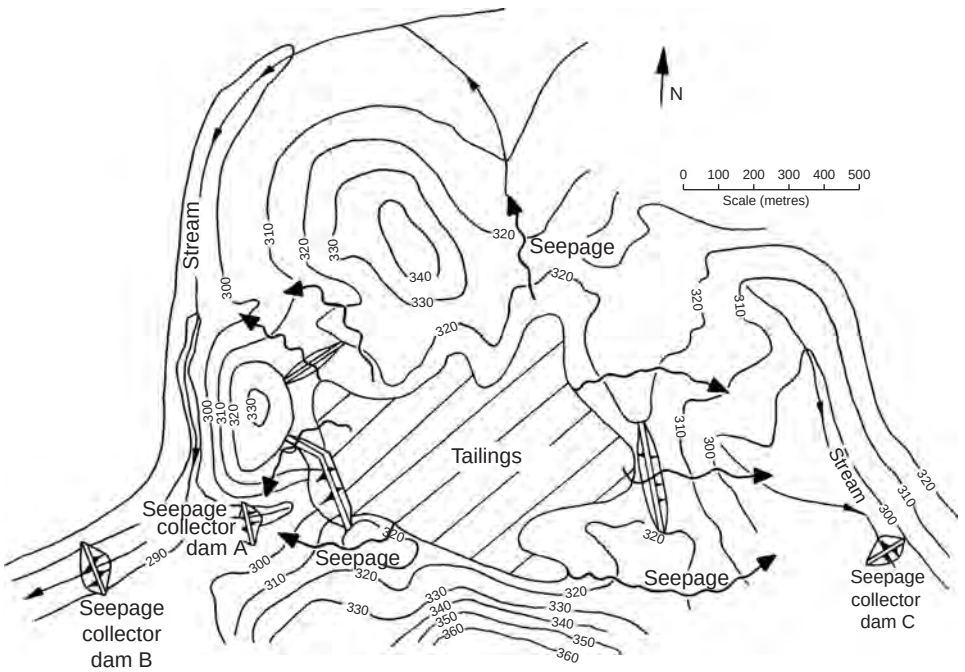


Figure 19.34 Cross-valley impoundment at the head of a small stream.

According to Robinsky (1979, 1999) the final cone slope is ideally around 6° to limit erosion. The concept is that the greater slope allows storage of larger quantities of tailings. It is claimed that this results in overall reduction in costs, as the saving in embankment costs more than offsets the costs of thickening.

Ritcey (1989) gives details of two mines where the method has been used. Blight and Bentel (1983) discuss the use of thickened tailings to increase the slope of tailings on a conventional dam to increase the storage capacity. They also give a method for estimating the stable slope of the tailings based on the yield point stress of the slurry.

The Robinsky method is a relatively uncommon, partly because it requires a very large area, and it is difficult to control exactly where the tailings deposit and the toe of the area is for run-off collection. It has been used successfully in at least one mine in Australia but in others it has been less successful.

Vick (1983) and Ritcey (1989) cite studies which have shown that some tailings can liquefy and flow at a slope of 6° and Vick cautions the use of the method in seismic areas. With advances in thickening technology some of the authors' reservation about the method are lessened. We have some concerns about surface erosion in other than arid areas and note that the method is not widely used, presumably on economic grounds.

The use of thickened tailings in more conventional tailings layouts is now common.

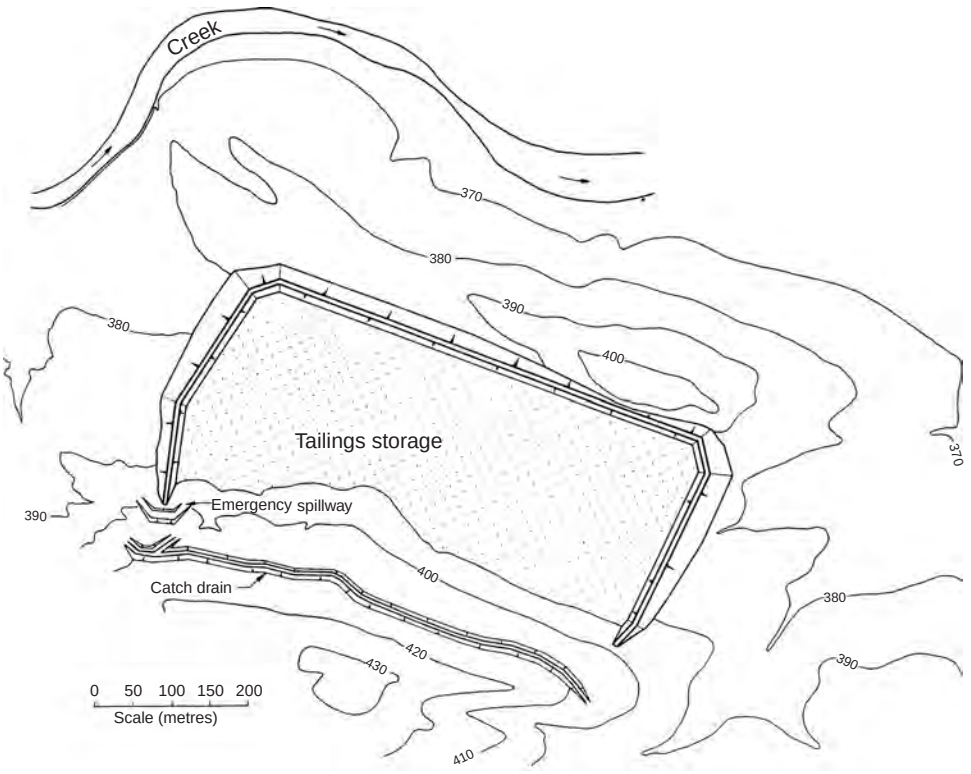


Figure 19.35 Side hill impoundment as was proposed for Ben Lomond tailings dam (Coffey and Partners 1982).

19.5.9.2 *Co-disposal*

Co-disposal of the coarse waste ('coarse reject') and tailings from coal washeries involves combining the waste streams and pumping them together to the storage facility. Williams and Gowan (1994) suggest there can be significant cost savings compared to the separate disposal of the coarse reject and the tailings and cite several Australian coal mines as using the techniques.

They indicate that the ratio of coarse to fine reject (by dry weight) ranges between 1.5 to 1 and 5.5 to 1, averaging about 3.5 to 1.

There is some breakdown of the coarse reject during pumping and further breakdown by chemical means or weathering (slaking) on the surface of the disposal area.

The combined materials are pumped at a solids content of 20% to 30%. The advantages claimed for co-disposal are (Williams and Gowan 1994):

Advantages claimed:

- May eliminate the need to build engineered tailings dams.
- Improved stability (shear strength) and the potential to use upstream construction.

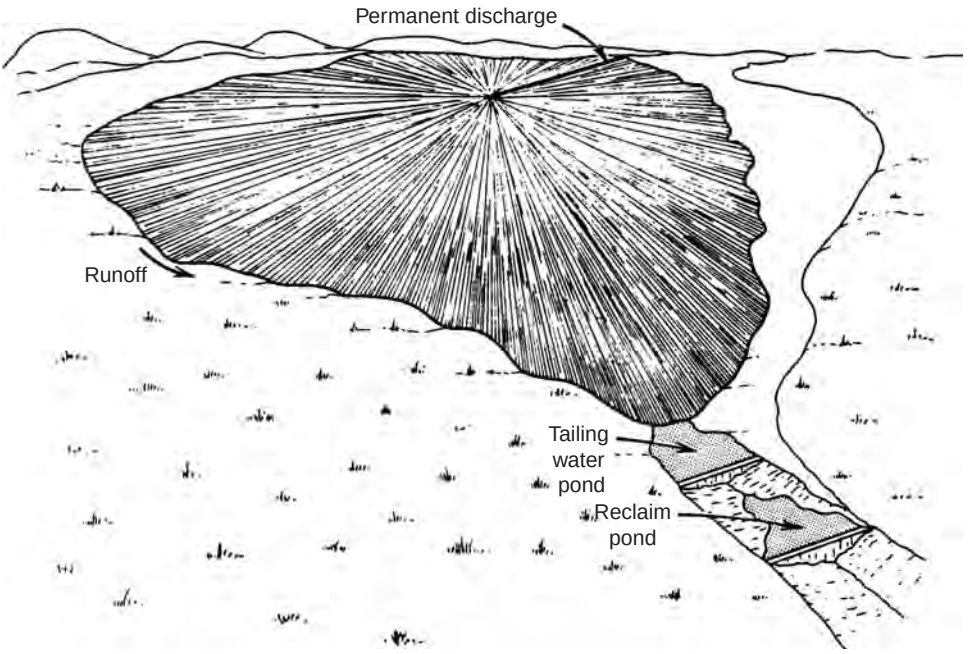


Figure 19.36 Thickened discharge disposal method (diagram from Vick, 1983, reprinted from Robinsky, 1979).

- Reduced need for trucks to dispose coarse reject.
- Reduced cost compared to the cost of the two separate waste streams.
- Rapid and enhanced dewatering with up to 40% more water recovery.
- Denser fill, with potentially up to 40% storage volume saved (because the tailings fill the voids in the coarse reject.
- Ready rehabilitation because of the denser, stronger fill, which can be trafficked soon after placement.
- Acid generation and spontaneous combustion of the coarse reject is less likely.

Disadvantages claimed:

- Blockage of pipelines.
- Pipe wear and replacement costs.
- Segregation of fines to form a wet tailings beach at the bottom of the co-disposal beach.

It would seem to the authors that co-disposal could be attractive where there is limited space for more conventional tailings storages and/or where acid generation and spontaneous combustion from the coarse reject is a problem.

However to date it seems it has been widely trialled but seldom used for on-going operations.

19.5.9.3 *Paste disposal*

Paste disposal involves dewatering the tailings to a solids content of 70–85% (water content 30–15%) using high efficiency cone thickeners or conventional thickening followed by partial vacuum filtration and paste mixing.

It falls between thickened tailings and belt filtration (see Section 19.5.9.4) in the amount of dewatering. For successful application, the tailings must have at least 15% finer than 0.020 mm so the water does not drain. The paste is often used as an underground mine backfill with cement added to increase the strength.

The paste behaves as a Bingham fluid and flow is initiated only after the pressure in the pipeline is sufficient to overcome the initial shear stress. The pressures to pump the paste are high (Brackenbush and Shillakeer, 1998, quote 2.25 kPa/m). Pumps are generally positive displacement, hydraulically operated. If it is stacked, its angle of repose is three to ten degrees. Claimed advantages and disadvantages are:

Advantages:

- Most process liquor is recovered by the thickening/filtration process, so less is likely to be lost to evaporation.
- Low permeability and capillary action limit acid generation in sulphidic tailings.
- Reduced groundwater pollution.
- The potential to eliminate containing 'dykes' or embankments.
- Possible elimination of liners because of the low permeability.
- Useful for underground mine backfill, particularly if cement is added.

Disadvantages:

- High capital and operating costs for dewatering of the tailings to paste consistency.
- Problems with performance of thickeners to give paste consistency.
- High pressures limit the distances paste can be pumped and costs are high.

Overall it is hard to see paste technology being attractive unless the paste is required for backfill in underground mines. Most of the advantages are available with thickened tailings (i.e. to say 60% solids content).

19.5.9.4 *Belt filtration*

Belt filters are used for dewatering of some tailings to such a consistency that they can be carried by truck for disposal in mine overburden dumps, either separately, or mixed with say coarse rejects. Figure 19.37 shows the principle of belt filtration.

The tailings are spread on a filter cloth supported on a drainage deck and are dewatered by gravity and the application of a vacuum to the underside of the drainage deck. The process is often aided by large doses of flocculants.

In general, the capital and operating costs of such operations are high and the success depends on having a uniform quality of tailings. To the authors' knowledge it has only been used where space or environmental constraints preclude other methods.

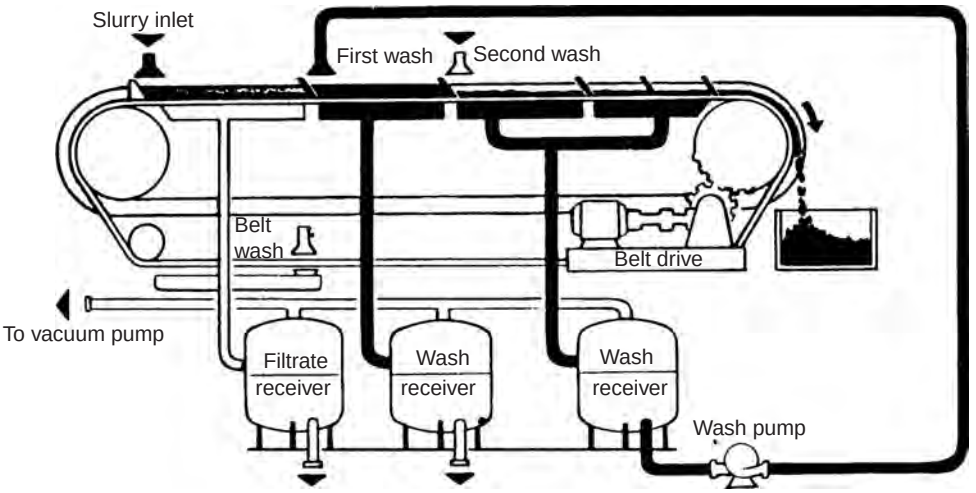


Figure 19.37 Principle of belt filtration (Willis, 1984).

19.5.9.5 *Disposal into open cut and underground mine workings*

An apparently attractive method of tailings disposal is to discharge the tailings into old open cut or underground mine workings. This presents a low cost method of disposal. However, there are several potential problems. In most open pits the area of the pit is relatively small and results in a subaqueous discharge environment with resultant low densities. This in turn leads to high compressibility and low strength. When the area is no longer in use, settlement occurs, leaving a water pond. Settlement can total several metres and take tens of years to complete. It may take some years before the tailings dry sufficiently on the surface to allow rehabilitation. Tailings discharged into underground mine workings will only settle to low densities and very low strength and will be susceptible to flowing out of the workings if for example another seam or ore body is removed from below the workings. There may also be a potential to contaminate the surrounding groundwater.

19.5.9.6 *Discharge into rivers or the sea*

There are some notable examples of this e.g. Bougainville Copper and OK Tedi Mining in Papua New Guinea and the Rosebery Mine, Tasmania. In the latter case, tailings discharge has resulted in devastation of the river due to the chemical content of the tailings leachate water. In Bougainville and in the lower reaches of the OK Tedi River, considerable sedimentation has occurred resulting in encroachment of the river into adjacent land. The OK Tedi River case ultimately lead to a very large pay-out to the Papua New Guinea Government. As a result of these types of effects, generally speaking, disposal into a river is environmentally unacceptable. The issues are discussed in Vick (1983) and Ritcey (1989).

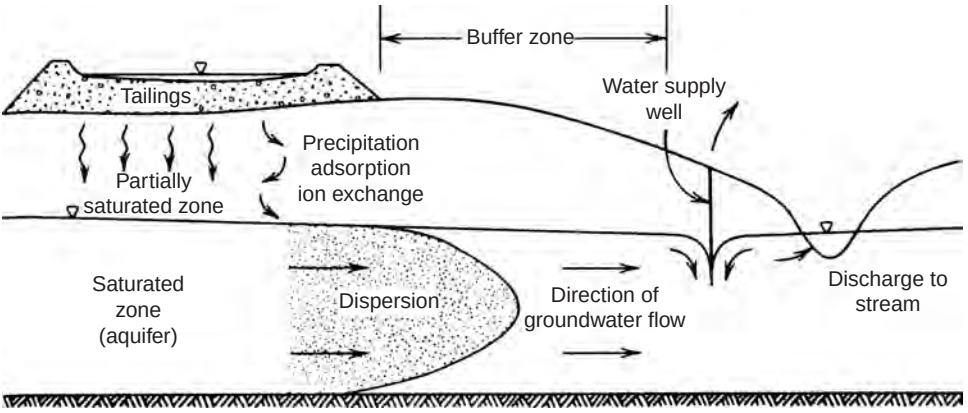


Figure 19.38 Groundwater flow and contaminant transport processes (Vick, 1983).

19.6 SEEPAGE FROM TAILINGS DAMS AND ITS CONTROL

19.6.1 General

As discussed in Section 19.2, many mine and industrial tailings have accompanying water or 'liquor' which contains dissolved salts, heavy metals and other residual chemicals from the mineralogical processes. If this liquor escapes to the surrounding surface water and groundwater in sufficient quantities, it can lead to unacceptable concentrations, making the water unusable for drinking and affecting aquatic life. Therefore, there is often an emphasis in the engineering of tailings dams on the estimation of seepage rates and, where these prove unacceptable, to the provision of measures to reduce seepage.

Seepage cutoff measures can be very costly and are often not as effective as the proponents would expect. They can affect the economic viability of a mining project and certainly the profitability of the operation.

This section presents an overview of the measures which are available and their effectiveness in controlling seepage. In many cases it is more realistic to accept that tailings dams will seep regardless of the measures adopted and to design collection and/or dilution of the seepage to acceptable concentrations.

For a more detailed discussion on the topic readers are referred to the chapter by Highland in Vick (1983). Ritcey (1989) and Fell *et al.* (1993) discuss chemical and geochemical aspects.

19.6.2 Principles of seepage flow and estimation

Many tailings storages (or 'dams') will be constructed on relatively flat land with a deep existing groundwater table. This situation is discussed by Vick (1983) and Figures 19.38 and 19.39 are reproduced from his book.

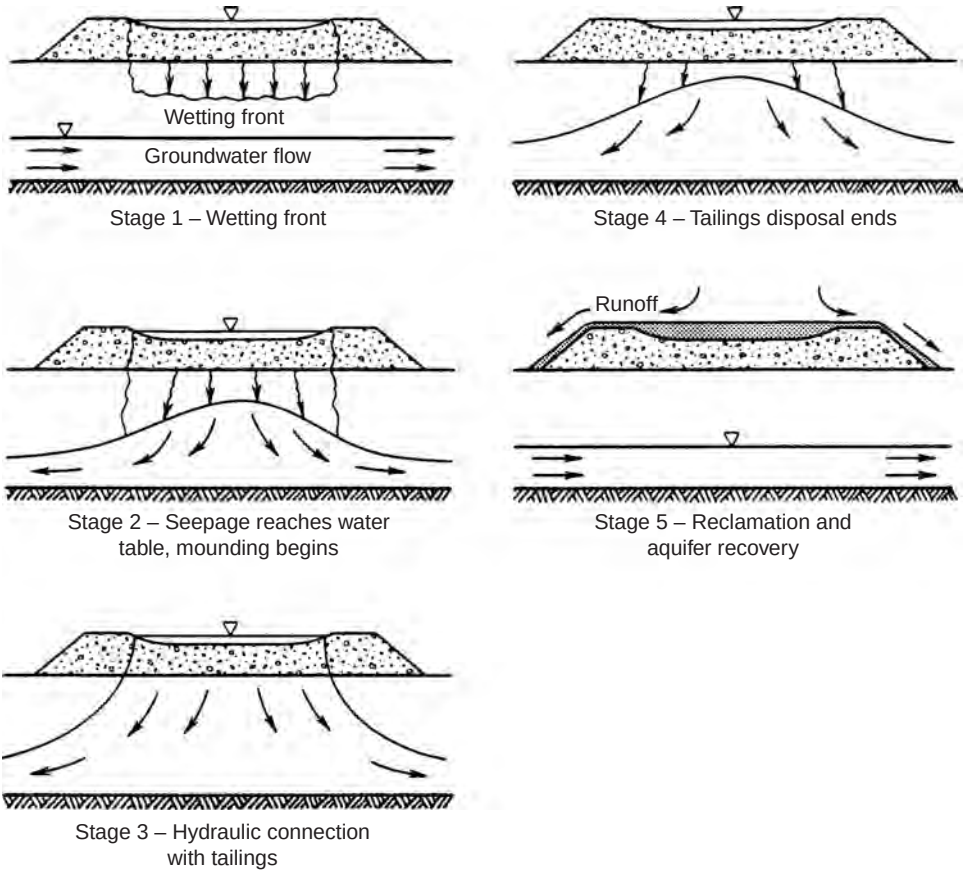

Wetting front
Groundwater flow
Stage 1 – Wetting front
Stage 4 – Tailings disposal ends
Runoff
Stage 2 – Seepage reaches water table, mounding begins
Stage 5 – Reclamation and aquifer recovery
Stage 3 – Hydraulic connection with tailings

Figure 19.39 Stages in seepage development (Vick, 1983).

The following should be noted:

- The rate of seepage flow will be dependent on the permeability of the tailings, the underlying soil and rock, climate, pond operation etc.
- Contaminants in the seepage water will not all join the groundwater. Much will be adsorbed in the foundation soil and rock. Hence, contaminant load does not equal seepage flow rate x contaminant concentration in the storage.
- Further reduction of contaminant concentration may occur in mixing with stream flows.
- It is contaminant concentration in ground and surface water which is generally critical, not the total quantity. Hence adsorption, dispersion and dilution can result in acceptable water quality in streams or well points, even though the original contaminant levels in the storage may have been unacceptable.

It is important to realise that, in many cases, a partially saturated flow condition will exist in the foundation at least at the start of operations, and possibly on a permanent basis if the tailings permeability is low compared to the foundation permeability. Figure 19.39 shows an example of the stages in the development of seepage.

Note that flow in the tailings in Stages 1 and 2 will be essentially vertical, will not emerge at the toe of the embankment, will be virtually unaffected by any foundation treatment such as grouting and will not be intercepted by drains at the toe of the dam.

It may take years for the seepage mound to rise to connect to the tailings (Stage 3) or it may never happen.

Figure 19.34 shows an example of a tailings storage which has been constructed at the head of a valley. In this case the final development will consist of several embankments. When estimating seepage from such a storage it is important to remember that seepage will occur under each of the embankments and depending on the base groundwater levels, into the hillsides adjacent to the embankments. Hence, in Figure 19.34, seepage will occur to the west, north and east but not to the south where natural groundwater levels are higher than the storage. Note that the groundwater does not always mirror the topography and may be affected by local variations in geology, e.g. permeable dykes.

It is the authors' experience that inexperienced engineers and geologists will either forget completely that seepage will occur in all directions from the storage or at least apply an excessive amount of the site investigation effort and analysis, to the seepage which will flow through and beneath the main embankment.

From the examples shown in Figures 19.38 and 19.39 and 19.34, it will be apparent that the assessment of seepage flow rates will involve:

- Knowledge of the permeability of the tailings, as these tailings are commonly part of the seepage path. In many cases they may control the seepage rates.
- Knowledge of the permeability of the soil and rock underlying the storage and surrounding the storage. In Figure 19.34 it would be necessary to be able to model the whole of the area between the streams, necessitating knowledge of rock permeabilities well beyond the storage area.
- Modelling of the seepage, usually by finite element methods, which may involve several section models and/or a plan model. This modelling should account for the development of flow as shown in Figure 19.39 and not just model an assumed steady state coupled flow situation, i.e. the storage and groundwater coupled as in Stage 3, Figure 19.39.

19.6.3 Some common errors in seepage analysis

Finite element seepage models are readily available and commonly used to estimate seepage from tailings storages. It is the authors' experience that the users of such programs sometimes fail to understand the actual boundary conditions which will apply or ignore details which are important to estimates. The following are some examples, which assume that the tailings have lower permeability than the underlying rock:

(a) Assuming saturated coupled flow, as shown in Figure 19.40(b). This assumption results in an underestimation of the rate of seepage compared to the correct conditions shown in Figure 19.40(a).

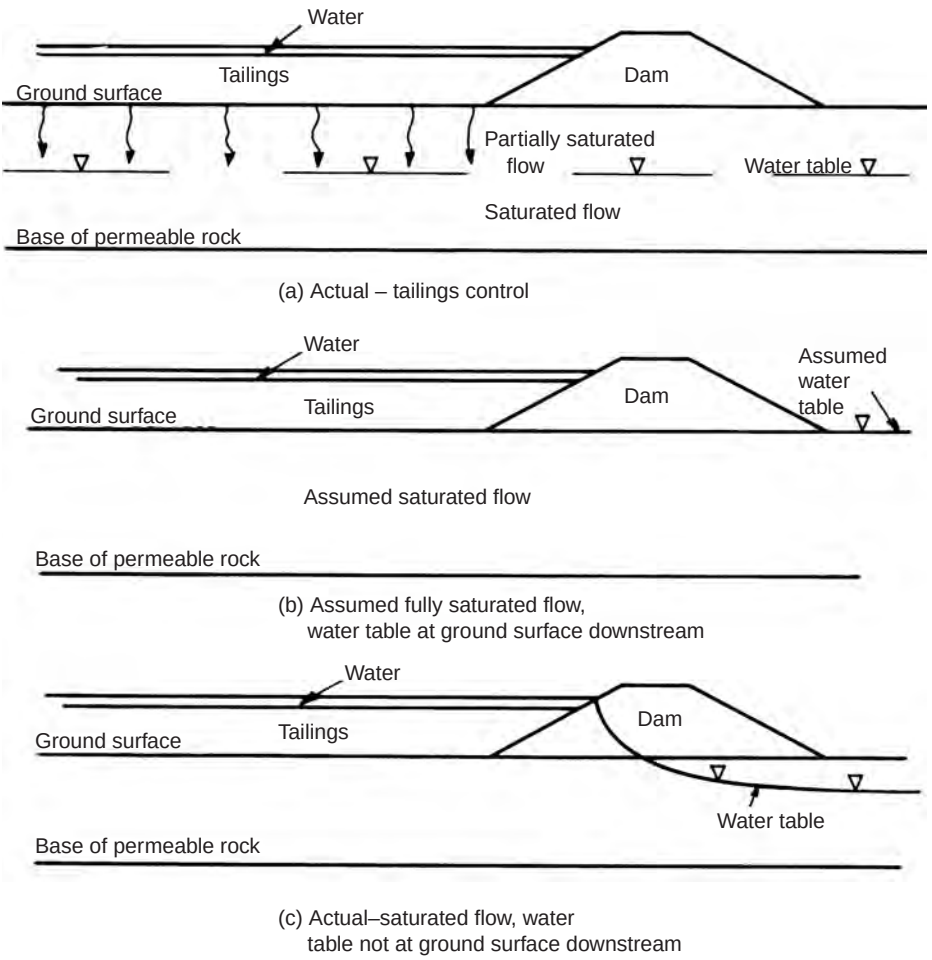


Figure 19.40 Analysis of seepage under a tailings dam.

(b) Assuming saturated flow as shown in Figure 19.40(b), with the water table at ground surface downstream, when flow is insufficient to result in such a high water table downstream (i.e. Figure 19.40(c) is more correct). This assumption results in an underestimation of seepage rates but, more importantly, seepage will not emerge at the downstream toe of the dam, so will not be able to be collected or monitored at the surface

(c) Failing to model and account for seepage through hills surrounding the dam, e.g. ignoring seepage to the north and east in Figure 19.34. This can result in significant underestimation of seepage rates.

(d) Assigning incorrect boundary conditions to the model as shown in Figure 19.41(b). This results in underestimation of seepage rates, and may lead to an overestimation of the amount of seepage which may emerge at the toe of the dam.

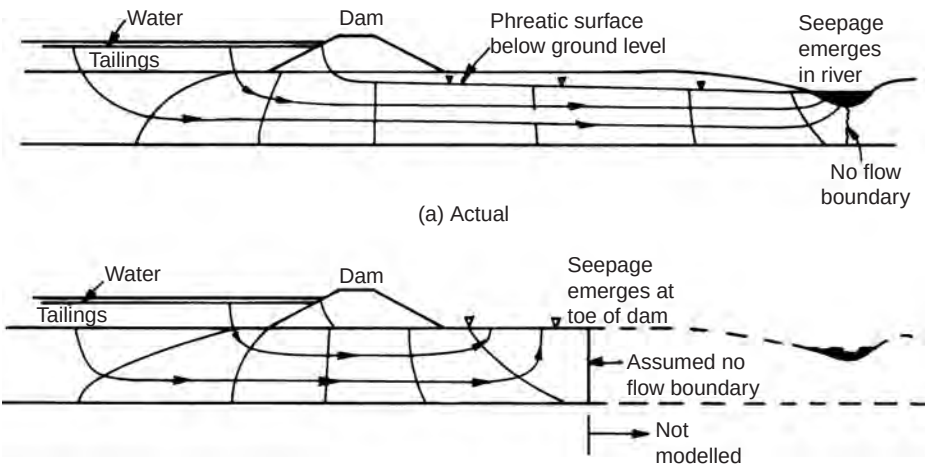


Figure 19.41 Incorrect boundary conditions for analysis of seepage under a dam.

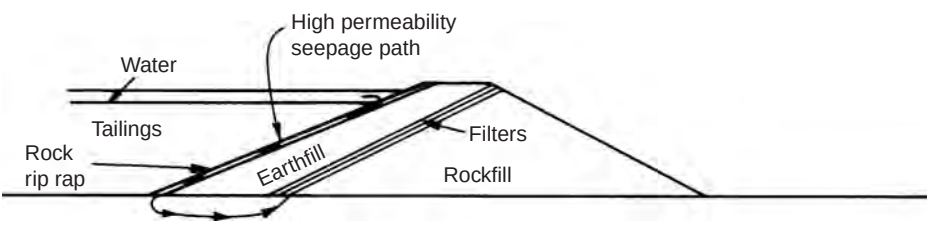


Figure 19.42 Seepage through rock rip-rap bypasses low permeability tailings.

(e) Failure to model permeable zones in contact with surface water, e.g. rock rip-rap zones on the upstream slope as in Figure 19.42 and high permeability alluvial, colluvial or lateritised soil zone on the surface as shown in Figure 19.43(a). In both cases, failure to model the high permeability zone can lead to significant underestimation of the seepage rate.

(f) Use of incorrect permeability in the foundation e.g.:

- Using the results of flow-in type permeability tests in soil and weathered rock, where smearing and/or blocking of fissures and joints yields lower than actual permeabilities.
- Use of permeability values obtained by methods which are unable to detect major leakage paths, e.g. from boreholes in deeply lateritised areas where drainage features are often localized and near vertical.
- Failing to allow for the variability of tailings permeability – as discussed above.

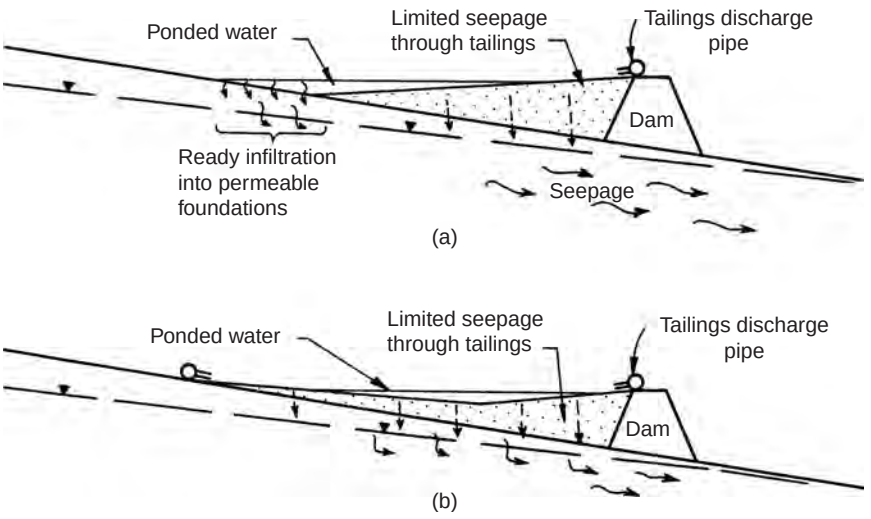


Figure 19.43 Controlled placement of tailings: (a) tailings not covering foundation; (b) tailings covering foundation.

(g) Incorrect estimation of travel times for contaminant fronts, e.g.:

- Use of porosity instead of storage coefficient in estimating flow velocities.
- Use of unrealistic values (too high) of storage coefficients in jointed rocks.
- Failure to allow for preferred seepage paths, e.g. high permeability, jointed or closely jointed zones.
- Failure to consider adsorption, dispersion, etc.

19.6.4 Seepage control measures

Measures to control seepage from tailings storages include:

- Controlled placement of the tailings
- Foundation grouting
- Foundation cutoffs
- Clay liners
- Geomembrane liners,
- Under-drains and toe drains.

19.6.4.1 *Controlled placement of tailings*

In many cases, the most cost effective way of controlling seepage will be to place the tailings so that the base of the storage is blanketed. Figure 19.43 shows this effect with the tailings forming an effective blanket in (b) but not in (a) where water is in direct contact with the foundation.

In theory, provided the tailings are of low permeability, they will form as effective a liner to the storage as can be achieved by a compacted clay liner. Vick (1983), for

example, shows that tailings with a permeability of 10^{-8} m/sec are as effective as a 2 foot (0.6 m) thick clay liner with a permeability of 10^{-9} m/sec.

It should be remembered that rock in the upper 10 m to 30 m of most foundations has a permeability between 1 Lugeon and 20 Lugeons, or 10^{-7} m/sec to 2×10^{-6} m/sec. Most naturally occurring soils will have a similar or higher permeability. Since tailings slimes (even from non oxidised ore) are likely to have a vertical permeability of less than 10^{-7} m/sec, the tailings will often be less permeable than the underlying soil and rock. If the tailings are from oxidised ore or from washeries (such as coal, bauxite, iron ore) they are likely to have a permeability of the order of 10^{-7} m/sec to 10^{-9} m/sec or less. Clearly, in these cases, covering the storage with the tailings will be an inexpensive and effective way of limiting seepage.

The effectiveness of the tailings as a 'liner' is dependent on placement methods. If tailings are placed sub-aerially and allowed to desiccate, lower permeabilities will result from the drying, provided cracking does not occur. If placed sub-aqueously lower densities and higher permeabilities are likely to result.

A potential difficulty with using tailings as a liner is that the coarser fraction of the tailings tends to settle out more quickly than the fine (or slimes) fraction. Hence a 'beach' of sandy tailings often occurs near the discharge point and if water is allowed to cover this area subsequently it can allow local high seepage rates. This can be overcome by using thickened discharge which inhibits segregation, by shifting the tailings discharge points from one end of the storage to the other, placing slimes under the beach area and/or by using a liner or seepage collector system under the sandy area.

Seepage can also occur along the contact between tailings and embankment if rock rip rap is used (Figure 19.42). Another problem is that it can be difficult to avoid water ponding against the storage foundation, particularly early in the storage operating life.

19.6.4.2 *Foundation grouting*

As discussed in Chapter 18, grouting is not particularly effective in reducing seepage, except in high permeability rock.

In a project on which the authors were involved, the grouting of a 5 km long dam foundation to a depth of about 25 m on average, would have reduced the estimated seepage by only 1%; nearly all of this in a relatively small portion of the foundation affected by faulting and with an un-grouted permeability of the order of 100 Lugeons.

It will be seen from the above discussion, that it is unlikely that grouting of tailings dam foundations can be justified on the grounds of reducing seepage. It may be justified on other grounds, such as reducing potential internal erosion in weathered rock, or where the high permeability zones can be identified from geological information and only these zones are grouted. It is usually not possible to do this.

19.6.4.3 *Foundation cutoffs*

For tailings storages constructed on soil foundations, particularly sand or sand and gravel, a significant reduction in seepage may be achieved by construction of an earthfill cutoff or a slurry trench cutoff wall as discussed in Chapter 10.

These cutoffs are of high cost and applicable only in critical situations and where ground conditions allow, i.e. generally soil. They may be applied to extremely weathered rock, e.g. lateritised highly permeable weathered rock.

19.6.4.4 *Clay liners*

Clay liners can be an effective way of reducing the seepage from a tailings storage. For example, if a tailings storage is located in a highly permeable area over sand, and the tailings are moderately high permeability, a clay liner may well be appropriate.

There are some practical aspects which should be considered in the provision of clay liners:

- The permeability of the clay depends on the soil available. For many naturally occurring soils the compacted permeability across the liner will be of the order of 10^{-8} m/sec to 10^{-9} m/sec. In the authors' experience, few soils have permeability as low as 10^{-10} m/sec.
- The permeability is affected by the compaction water content and density. To achieve a low permeability, the soil should be compacted to a density ratio of 98% of standard maximum dry density, at a water content of between −1% of optimum to +2% of optimum. The permeability can be increased by an order of magnitude by compacting dry of optimum – see Lambe and Whitman (1981).
- The thickness of the clay liner may not be particularly critical, depending on the particular circumstances. It is better to have a relatively thin (say 0.6 m) high quality layer (i.e. good selection, good compaction control) than a thicker, less controlled layer.
- The clay liner is susceptible to cracking on exposure to the sun which can increase its permeability by orders of magnitude. This is particularly critical on sloping sites, where the liner may not be covered by tailings for months after construction. On flat sites, covering the liner with, say, 150 mm of clean sand or silty sand, can act to prevent drying of the clay. On sloping sites the sand cover may be eroded by rainfall and may need to be held in place with a geotextile. As shown by Kleppe and Olson (1985), once cracked, the liner permeability will remain high, even if the cracks are apparently closed by swelling on re-wetting.
- The permeability of clay liners was questioned in the early 1980s, when some researchers found that the permeability was increased by some orders of magnitude when particular organic leachates were passed through the clay. Later research has shown this is not a problem for water containing inorganic chemicals, even if at high concentrations. Since most tailings liquor would not contain organic leachate, the permeability measured in the laboratory should be a reasonable guide to its long term behaviour.
- If the foundation has openwork gravel or wide open joints, clay liners can be subject to 'sinkhole' development, i.e. erosion of the liner into the underlying foundation unless a filter is provided between the liner and the gravel to control the erosion or shotcrete is used to cover open joints.
- The clay liner could be potentially quite an expensive option, given the likely area to be covered. A readily available source of earthfill would be essential, if the total cost is not to be prohibitive. Once allowance for related activities such as surface

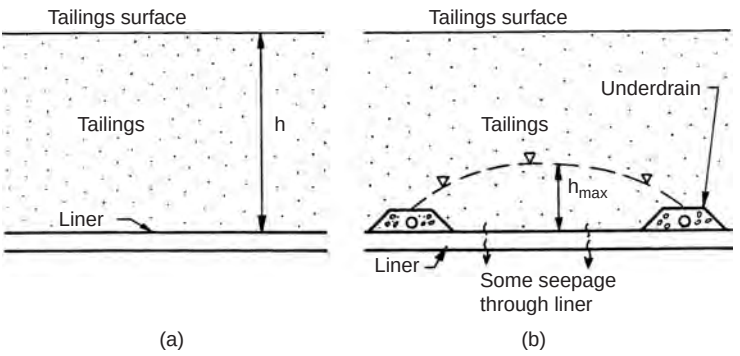


Figure 19.44 Comparison of head acting on clay liner: (a) Without underdrain; (b) With underdrain.

preparation to give a smooth contour, any protection against drying and erosion and underlying filter/drain zone, the total cost is likely to be a significant factor to be considered.

- The clay liner should cover the whole of the tailings storage. Use of a liner as a minimal 'upstream blanket', such as might be used in a conventional water storage dam, is unlikely to result in significant reduction in seepage quantities.
- There will be significant seepage through a clay liner. For example, a 0.6 m thick clay liner with permeability of 10^{-9} m/sec would discharge 90 m^3/day over an area of 1 km square under unit gradient.
- The naturally occurring clays in a storage area are unlikely to have a low permeability unless they are excavated and re-compacted. *In situ* they are likely to have a permeability of the order of 10^{-5} m/sec to 10^{-6} m/sec due to the presence of root holes, fissures etc.

Unfortunately some regulatory authorities have unrealistic views in regards to clay liners for tailings dams and may request provision of a liner, jeopardizing the viability of a project, when the geochemistry is such that seepage of an unlined storage is not an environmental hazard.

19.6.4.5 *Under-drains*

Drains may be provided under the tailings as shown in Figures 19.28, 19.29 and 19.44, with or without a clay liner. The drains act to attract the seepage water and discharge it to a collector system, often for recycling to the process plant.

As shown in Figure 19.44 the under-drains may reduce the head on the liner and reduce the seepage. However, the amount of seepage bypassing the collector drains, and the efficacy of the under-drains in reducing pressures on the liner is dependent on many factors, as follows:

- The spacing of the drains.
- The vertical and horizontal permeability of the tailings.
- The permeability of the liner. Note that there is a net head on the liner between the drains and, even within the drain, water flowing on the liner will percolate

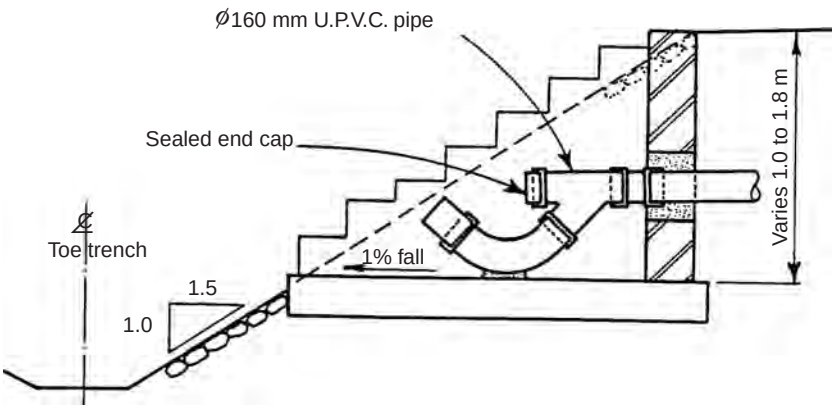


Figure 19.45 Outlet detail for drain to prevent clogging by oxidation (Lyell and Prakke, 1988).

through the liner. If the groundwater level is below the liner, the hydraulic gradient through the liner will still be at least 1, giving significant seepage.
- The efficiency of removal of water from the drain system.

If no liner has been provided, and the under-drains are laid on the natural ground, there will still be significant seepage into the ground because while the drain has a permeability of say 10^{-5} m/sec the ground will have a significant permeability, e.g. 10^{-6} m/sec to 10^{-7} m/sec (if normal weathered rock).

Apart from these design problems, there are some practical aspects:

- The under-drains must be designed to act as filters (as detailed in Chapter 9) to the tailings, or they will rapidly become blocked. Tailings are particularly erodible and will readily clog a poorly designed filter.
- Geotextiles have been used to construct the drains. However, as outlined in Scheurenberg (1982) and Bentel *et al.* (1982), the geotextile should not be exposed directly to the tailings or it will clog. They overcame this problem by covering the geotextile with a layer of filter sand. There is however a chance that the geotextile will still clog with deposited oxides from the seepage water and their use is not recommended.
- The under-drains are susceptible to contamination by dry windblown tailings, before they are covered with tailings.
- The seepage water collected in the drains has to be collected and pumped to storage for use in the process plant. This involves expensive collection and pump systems which have to be maintained.
- The filters and outlet pipes can be clogged by oxidation products from the tailings. Figure 19.45 shows an outlet system used by Lyell and Prakke (1988) to overcome this problem.

19.6.4.6 *Synthetic liners (geomembranes)*

Synthetic liners or geomembranes are used to line many hazardous waste facilities, often with the provision of drainage layers beneath the first membrane, with a second membrane or clay liner to collect any leachate which leaks past the first. Figure 19.44 shows such a system for a proposed uranium tailings storage in New Mexico, USA.

19.6.4.7 *Geomembrane liners*

However, these liners are expensive and are not appropriate for most tailings disposal situations. There are notable exceptions, e.g. synthetic liners have been used for tailings storages in the Alcoa Alumina Refinery at Kwinana, where the storages are underlain by sands which are used as an aquifer for water supply. In many applications on sloping sites they would be impracticable, because of the need to provide a well graded base, free of irregularities which may penetrate the liner.

It is understood that permeabilities of the order of 10^{-12} m/sec to 10^{-14} m/sec are applicable. A 5 mm thick liner with permeability of 10^{-13} m/sec and 15 m of water head on it would give a leakage rate of 25 m^3/day per square kilometre, i.e. significant leakage will occur. A common problem is susceptibility to deterioration in sunlight, requiring the liner to be covered.

As for clay liners, unfortunately some regulatory authorities have unrealistic views in regards to liners for tailings dams and may request provision of a liner, jeopardizing the viability of a project, when the geochemistry is such that seepage of an unlined storage is not an environmental hazard.

Geomembrane design and construction is a specialist area and expert advice should be sought if such a liner is to be used.

19.6.5 Seepage collection and dilution measures

From the above discussion, it will be apparent that no matter what measures are adopted there is going to be some seepage from tailings storages. This seepage will probably be greatest during operation of the storage, particularly early in the operation when there is little blanketing effect from tailings. It will be exacerbated if water is allowed to pond over the tailings and adjacent to the natural ground. Seepage will continue after shutdown, even when the tailings are covered, because infiltration of rainfall will occur. Granted this is the case, it will often be more practicable to limit expenditure on seepage 'control' (or 'prevention') measures described above and design measures to collect the seepage. This may then be pumped back to the storage or the process plant, or diluted with surface runoff before release at acceptable concentrations. Measures which may be taken include:

19.6.5.1 *Toe drains*

A drain may be provided at the downstream toe of the embankment, to collect seepage which emerges at that location.

These drains can be reasonably successful in intercepting seepage, but only if the seepage naturally emerges in this location. In many cases, the flow rates will be such that the phreatic surface stays below the level of the drain. Even when the seepage

is sufficient to raise the phreatic surface to flow to the drain, much may still bypass by flowing beneath the drain. Ideally the drain has to penetrate to a low permeable stratum, but this is often not practicable.

Such drains may also intercept surface runoff from the downstream face of the dam and groundwater from downstream (if the water table is high) and, if the seepage is to be returned to the dam or process plant, may exacerbate water management problems if a 'no release' system is being operated.

19.6.5.2 *Pump wells*

Seepage can be collected by constructing water wells into pervious strata downstream of the tailings embankment and pumping from these back into the storage or to the process plant.

Such a well system can be reasonably successful in intercepting seepage but there are some disadvantages:

- The pumps lower the piezometric pressures downstream of the storage, so gradients and seepage rates from the storage may be increased.
- The wells also attract water from downstream and so may also add to water management problems in no release operations.
- The wells have to be pumped continuously to be effective, with all the associated costs.
- The well screens and pumps are susceptible to corrosion and blockage and require maintenance and periodic replacement.
- It is unlikely that it is practicable to operate the wells after shutdown of the storage, so another method may be needed to handle long term seepage.

19.6.5.3 *Seepage collection and dilution dams*

In many cases, a practical way of collecting seepage from tailings storages will be to construct a seepage collector dam or dams. Figure 19.34 shows such a system.

The seepage collector dams may be located sufficiently close to the storage to collect the bulk of seepage, but not too far away so as to limit the external catchment, e.g. Dam A on Figure 19.34. In this case water would normally be pumped back into the dam or to the process system.

Alternatively, one may deliberately locate the collector dam sufficiently far downstream to ensure that the runoff from the catchment to the dam is sufficient to dilute the seepage to acceptable water quality, e.g. Dam B in Figure 19.34.

Whether such an approach is acceptable will depend on the particular circumstances for the tailings storage. For example, it may be unacceptable to have substandard water quality in the stream between Dam B and the tailings storage. Seasonal effects can also be important, e.g. if there is a prolonged dry season, water may pond in the stream and concentration of contaminants may occur.

The authors' view is that, in many cases, the catch dam with pump-back or dilution may be far more appropriate than expensive measures to control the seepage. From the authors' experience, too many engineers and regulators have an over optimistic view of the efficacy of these seepage control measures, or an unrealistic view of the costs a mining operation can reasonably bear to construct such measures.

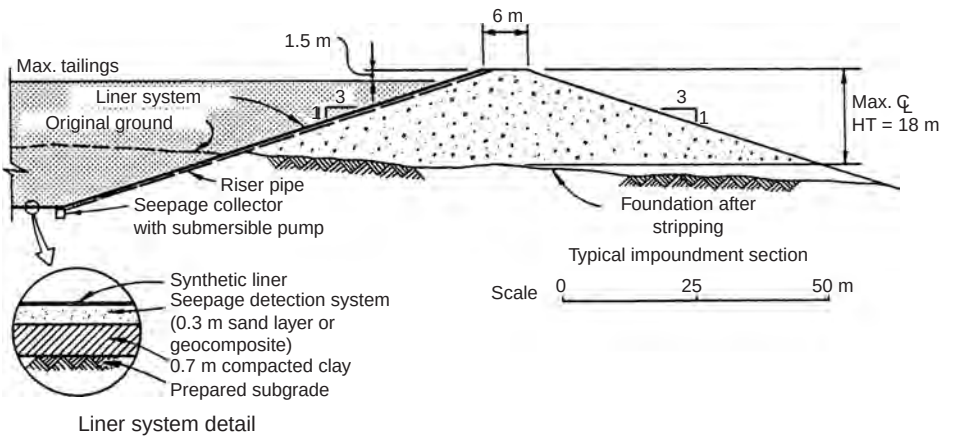


Figure 19.46 Proposed uranium tailings storage (Griffin, 1990).

19.6.6 Rehabilitation

After operation, tailings storages must be designed to contain the tailings indefinitely and minimise long term impact on the environment. Factors are:

- long term stability of the 'dam'
- long term erosion
- long term effect on groundwater and surface water
- return of area to productive use.

19.6.6.1 *Long term stability and settlement*

Provided water is drained off the storage, stability and settlement should not be a problem because seepage piezometric pressures are reduced. It is important to ensure that storm runoff cannot overtop the 'dam', or erosion from creeks etc. cannot erode the toe of the 'dam'.

Ponding of water on the tailings storage is likely to occur if tailings have been deposited sub-aqueously with resulting low settled density and high compressibility. Figure 19.47 shows the sort of problem which can arise.

In many cases the presence of a water pond does not create difficulties, but if the water is contaminated it may be unacceptable. In this event it will be necessary to construct a spillway as shown or to fill the depression with waste rock, more tailings etc. However this filling will in itself induce additional settlement. Mounding of the tailings and cover prior to shutdown can help alleviate the problem. If tailings are placed sub-aerially and well desiccated, they will have dried to a high strength, low compressibility landfill and, provided the surface is adequately contoured, water ponding will not be a problem.

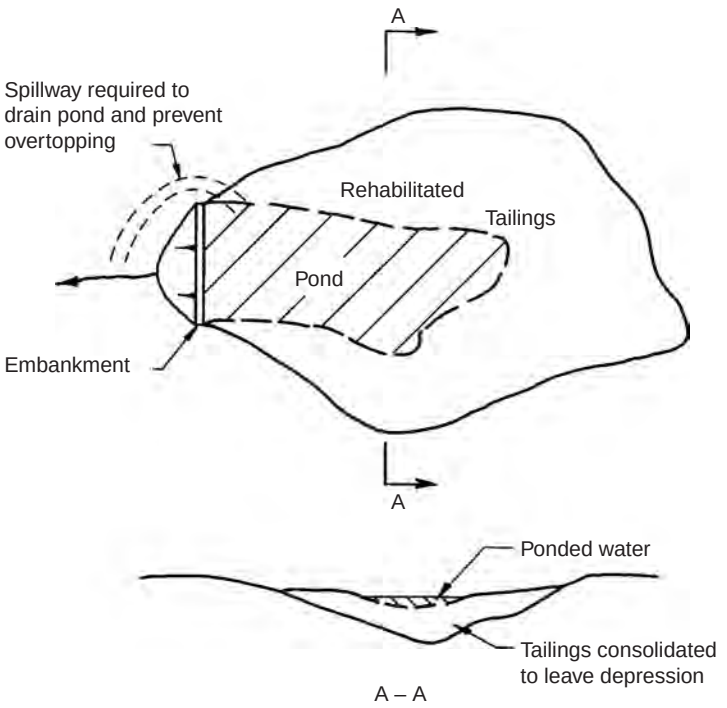


Figure 19.47 Potential ponding after operation ceases.

19.6.6.2 *Erosion control*

This can be a major issue. Wind erosion, as well as water erosion, has to be considered. The embankment side slopes are best covered by rock if this is available, otherwise flat (flatter than say 3H to 1V) slopes with good vegetation are required. This is often impossible to achieve on a year round basis.

The tailings surface may be covered with soil, waste rock or a combination of the two and vegetated to control erosion and limit infiltration. Some tailings can be successfully revegetated without the need for cover. Cement stabilization has also been used. Figure 19.49 shows some examples.

It is often necessary to carry out trials during operation to determine what measures will be successful.

Blight (1988) and Blight and Caldwell (1984) describe measures taken to alleviate erosion from abandoned gold tailings dams. Ritcey (1989) also discusses the design of cover. Forrest *et al.* (1990) and Corless and Glenister (1990) also discuss erosion control measures.

19.6.6.3 *Seepage control*

In many cases the deposited tailings will have contaminants trapped in the accompanying water when operations cease. These contaminants will continue to seep from

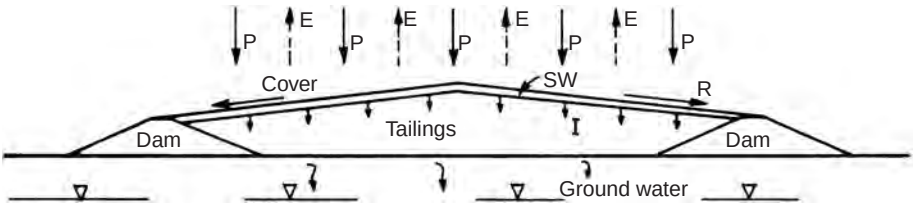


Figure 19.48 Factors involved in estimating infiltration through cover over tailings.

the tailings as they consolidate, and the water will infiltrate through the cover leaching contaminants as it passes through the tailings.

As was the case during operation, this may or may not be a problem, depending on the concentration of contaminants that reaches groundwater wells or streams in the vicinity. If the contaminants are likely to be a problem, measures will have to be taken to limit infiltration. This can be achieved by redirecting external catchment flows (see Figure 19.35), contouring the surface of the tailings to encourage runoff, avoiding ponding of water as shown in Figure 19.47, and encouraging transpiration by planting vegetation on the tailings.

It is unrealistic to consider that a 'clay cover' can be provided which will 'seal' the tailings. In the first place, the clay will have a finite permeability even if well compacted and, in any case, the clay may be difficult to compact, because the tailings do not form a strong base. The permeability of the cover will be increased by cracking due to desiccation and settlement and penetration by roots and animals.

It is more realistic to design the total system on the assumption that there will be long term seepage, and if the resultant contaminant concentrations are too high, to design seepage collector dams to allow dilution to acceptable concentrations prior to release.

Figure 19.48 shows the components involved in assessing the infiltration into tailings. In this diagram P = precipitation (rainfall); E = evapotranspiration; R = runoff; I = infiltration; SW = soil water stored in the 'cover' and $I = P\text{-}E\text{-}R\text{-}\Delta(SW)$.

Figure 19.49 shows some alternative 'covers' which may be used. These are in increasing degree of sophistication and increasing cost from (a) to (d). It should be emphasized immediately that it is seldom practicable to use such complicated and costly multiple layer systems as shown in (d) for tailings (or waste rock). Such systems may be used in covering hazardous waste landfills.

In these figures:

- The vegetation controls erosion, promotes evapotranspiration and enhances the appearance and use of an area.
- Topsoil promotes the vegetation. Note that in some cases vegetation can be established directly on to tailings.
- Soil rockfill is to control erosion and protect the underlying compacted clay from desiccation (in d).
- Compacted clay is a 'barrier' to infiltration (in reality it only reduces infiltration since it will have a finite permeability).

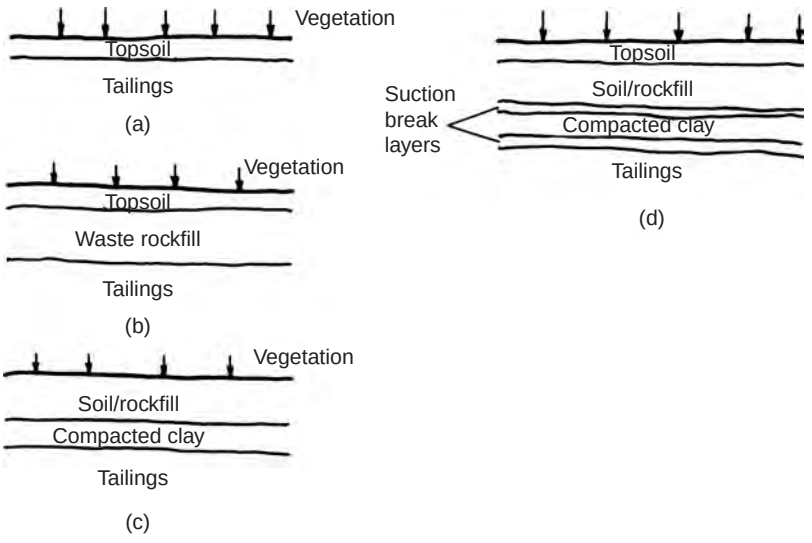


Figure 19.49 Some alternatives for covering tailings.

– Filter layers are to prevent migration of fines into the drainage layer (a).
– Suction break layers are sand layers which assist in controlling desiccation and hence cracking of the compacted clay and help prevent transfer of salts and contaminants up through the compacted clay from the tailings.

The adopted design will depend on environmental constraints and availability of materials. Geotextiles and geomembranes may also be used, but are not common for tailings or waste rock dumps because they are too expensive for the large areas involved.

Accurate estimation of the rate of infiltration through the cover is difficult (almost impossible).

There are finite element and finite difference models which can in principle model the physical processes involved in infiltration through cover. Richards (1992) and Schroeder *et al.* (1983) describe such models, which rely on knowledge of the properties of each of the soil and rock layers in the cover, including saturated and unsaturated hydraulic conductivity (permeability) and the relationship between suction pressures and water content. These properties are very difficult to predict and vary by order of magnitude over normal ranges of compaction density and water content. The Schroeder *et al.* (1983) HELP model seems to have fairly wide acceptance. Readers are encouraged to read the latest literature for available methods.

There are other practical problems associated with the analysis of infiltration through cover. These include:

– Differential settlement of the tailings or waste rock or drying of the cover can lead to cracking, with a resultant large increase in effective hydraulic conductivity. This is virtually impossible to predict for cover placed over highly compressible

tailings. The use of geotextiles or geogrids to control cracking would lead to a greater degree of confidence in the predictions, but again these all usually too expensive for use in tailings storage.

– Vegetation performs a very useful role in promoting transpiration. However the roots may penetrate the compacted soil layer leading to an increase in infiltration. Use of a silty sand or sand layer over the compacted soil, or a chemically treated zone to prevent root penetration may assist.

19.6.6.4 *Return of area to productive use*

In principle, it is desirable to return the area to productive use. Usually this can only be for farming or other low intensity usage not affected by long term settlement.

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Chapter 20

Monitoring and surveillance of embankment dams

20.1 WHAT IS MONITORING AND SURVEILLANCE?

ANCOLD (1976, 2003) give the following definitions:

Monitoring. The observing of measuring devices that provide data from which can be deduced the performance and behavioural trends of a dam and appurtenant structures, and the recording of such data.

Surveillance. The continuing examination of the condition of a dam and its appurtenant structures and the review of operation, maintenance and monitoring procedures and results in order to determine whether a hazardous trend is developing or appears likely to develop.

Figure 20.1 explains the concepts of surveillance.

Monitoring and surveillance should be carried out during the construction, first filling and operation of all large dams. Wherever possible groundwater measurements should be started before any major construction work is begun, preferably as soon as the site is selected and a firm decision is made to proceed with the design and construction.

There is a generally accepted principle that the level of monitoring and surveillance appropriate for a dam depends on the consequences of failure of the dam, whether the dam is being filled for the first time or is in general operation, and whether abnormal behaviour has been detected. This is discussed more in Section 20.3. The definitions of consequence of failure ratings are given in Section 10.2.3.

It should be remembered that the consequence of failure rating can change during the life of a dam. For example development downstream may raise the consequence of failure rating from low to high.

20.2 WHY UNDERTAKE MONITORING AND SURVEILLANCE?

20.2.1 The objectives

The objectives of monitoring are (ANCOLD, 1983):

- To provide confirmation of design assumptions and predictions of performance during the construction phase and initial filling of the reservoir.
- To provide during the operation phase of the life of the dam an early warning of the development of unusual and potentially unsafe trends in behaviour.

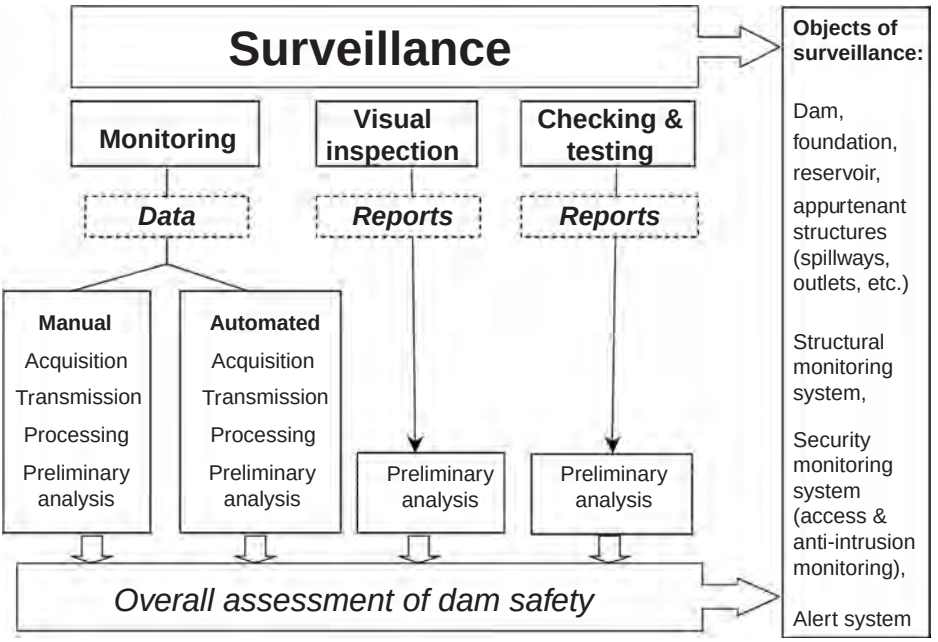


Figure 20.1 Concepts of dam surveillance (ICOLD, 2009, 2014).

- To provide data on behaviour of dams which may not conform with accepted modern criteria and warrant continuous and close monitoring as a guide to the urgency for introduction of remedial/stabilizing works or other measures.
- During raising or remedial/stabilizing works, which may need to be carried out with the storage full, close monitoring of structural/seepage behaviour is warranted to ensure that the additional loading introduced by the new works is applied in a manner which will not adversely affect the safety of the dam.

In addition to this may be added:

- To satisfy legal obligations of the duty of care.
- To provide data to allow developments in dam engineering: through better measurement of properties, e.g. rockfill modulus in CFRD, checking of analytical methods, e.g. displacements of CFRD face slab, and new construction materials, e.g. asphaltic concrete core.

When setting up a surveillance framework it is vital to ensure that:

- The inspection and monitoring program is planned by qualified experienced dam engineers and engineering geologists, taking account of the potential failure modes for the dam. A detailed potential failure modes analysis (PFMA) should be a required part of establishing and reviewing an inspection and monitoring program.

ANCOLD (2003), FERC (2005) and USBR (2012c) provide guidance on carrying out PFMA.
- The monitoring data is reviewed by qualified experienced dam engineers and engineering geologists in an ordered manner, so that unusual behaviour can be identified and appropriate action taken.
- The responsibilities of the owner, operator and government authority are clearly defined, with lines of communication established.

20.2.2 Is it really necessary?

That it is necessary to have a monitoring and surveillance system established, is highlighted by the number of dams which experience accidents and failure, often after many years of operation. This is discussed in some detail in ICOLD (1983), National Research Council (1983), Foster *et al.* (1998, 2000a), and Douglas *et al.* (1998, 1999).

ICOLD (1983a) studied the approximately 14,700 dams which qualified for the ICOLD register at that time. An extensive survey indicated that of these dams 1105 (7.5%) had suffered incidents and deteriorations of one or more type and 107 (0.7%) had failed. In some cases the dam had to be completely abandoned and in others the dam was repaired and brought back into use despite severe damage.

Figure 20.2 summarises the failure of dams and causes of failure of dams over 15 m high built from 1900–1975.

Tables 8.1, to 8.5 and Table 20.1 summarise the statistics of failures and accidents for embankment dams.

It can be seen that for embankment dams:

(a) Overtopping and internal erosion and piping are the main causes of failure.
(b) Slope instability is a relatively minor cause of failure but is a significant cause of accidents.
(c) About two-thirds of internal erosion and piping failures and about half of internal erosion and piping accidents occur on first filling or in the first 5 years of operation. However accidents and failures do still occur in older dams.
(d) About two-thirds of slope instability failures and half of instability accidents occur after 5 years of operation.
(e) The ratio of accidents to failures is quite different depending on the failure mode. In particular, slope instability and piping from embankment to foundation are much more likely than piping within the embankment to result in accidents compared to failures.
(f) The percentage of dams suffering accidents is not improving with time, but the percentage of failures is decreasing, reflecting improved technology of design, construction, monitoring and surveillance.
(g) More than 79% of all embankment dams are less than 30 m high. These suffer proportionally more failures than higher dams, but fewer accidents. This may reflect better design and monitoring and surveillance of the larger dams.

Douglas *et al.* (1998, 1999) showed that for concrete dams:

(a) Sliding, leakage and piping in the foundation are the main causes of failure. Accidents are most likely to be high recorded uplift or leakage in the foundation.

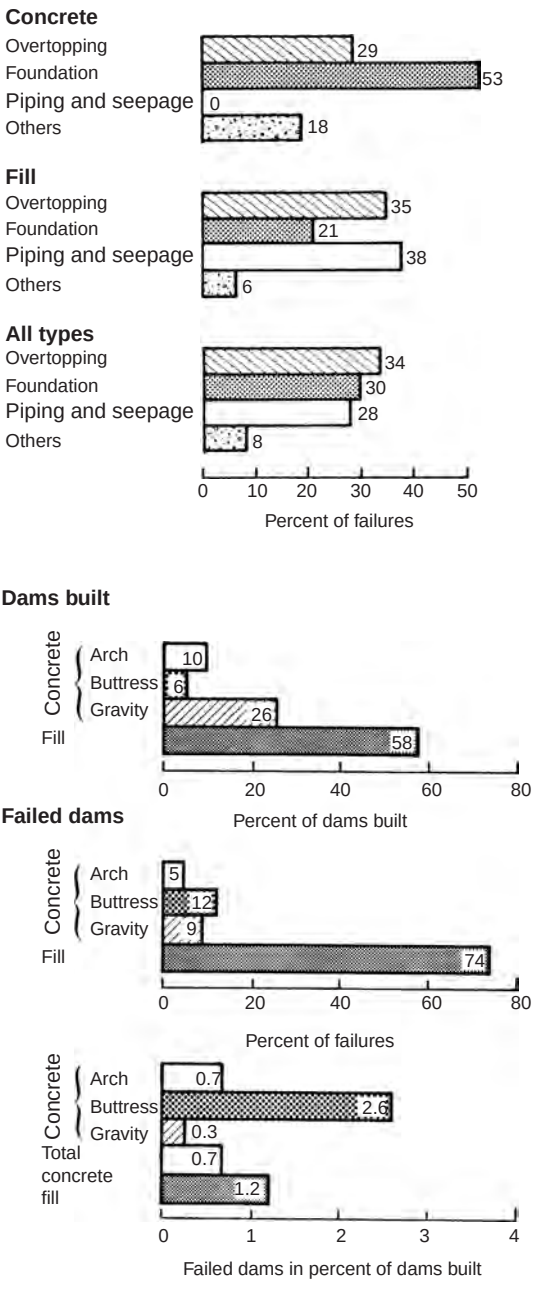


Figure 20.2 Summary of failures of dams and causes of failure of dams over 15 m high (1900–1975) (National Research Council 1983).

Table 20.1 Statistics of failure and accidents of embankment dams up to 1986 (from Foster *et al.*, 2000a).

	Average[(2)] *probability* $\times 10^{-3}$		*Timing of incident (%)*							
			Failure				*Accident*			
Failure mode	*Failure*	*Accident*	*DC*	*FF*	*<5 YR*	*>5 YR*	*DC*	*FF*	*<5 YR*	*>5 YR*
Internal erosion and piping										
– Embankment	3.5	6.7	2	48	14	36	0	26	13	61
– Foundation	1.7	6.2	5	20	50	25	0	30	24	46
– Embankment to foundation	0.2	2.1	0	50	50	0	0	20	27	53
Slope instability										
– Downstream	0.5	5.3	18	18	0	64	15	11	25	49
– Upstream	0.1	4.2	0	0	100	0	22	2	26	50

Note: DC = during construction; FF = during first filling; <5 YR = in the first 5 years of operation, not including first filling; >5YR = after the first 5 years of operation.

(b) Internal erosion and piping in the foundation is the main cause of failure for concrete dams on soil foundations.
(c) A large proportion of failures occur on first filling, or historic high reservoir levels. This is particularly the case for internal erosion and piping failures. Older dams are also represented in the failures.

From this information it is clear that dams are most likely to fail or have an accident on first filling or in the first 5 years, but that there are failures and accidents which occur in older dams. It is apparent that many accidents would have become failures if they had not been detected by monitoring and surveillance and some action taken.

20.2.3 Some additional information on embankment dam failures and incidents

Figure 20.3 shows the location of piping failures and accidents for piping in the embankment (Foster *et al.*, 1998).

Foster and Fell (1999b) showed that for piping not associated with conduits:

- Cracks associated with narrow cores and arching between rockfill shell zones are generally located at depths from one third to two-thirds of the height of the dams.
- Cracks and piping associated with broad changes in the abutment profile are generally 'located at depths of less than one third of the height of the dam'.
- Piping associated with small scale irregularities in the foundation generally occurs close to the foundation surface and usually in the lower half of the core.

Foster *et al.* (1998, 2000a) record that in 80% of failures by internal erosion and piping through the embankment, the reservoir was at or higher than the previous highest reservoir level. For the other recorded cases it was within 1 metre of the previous highest level.

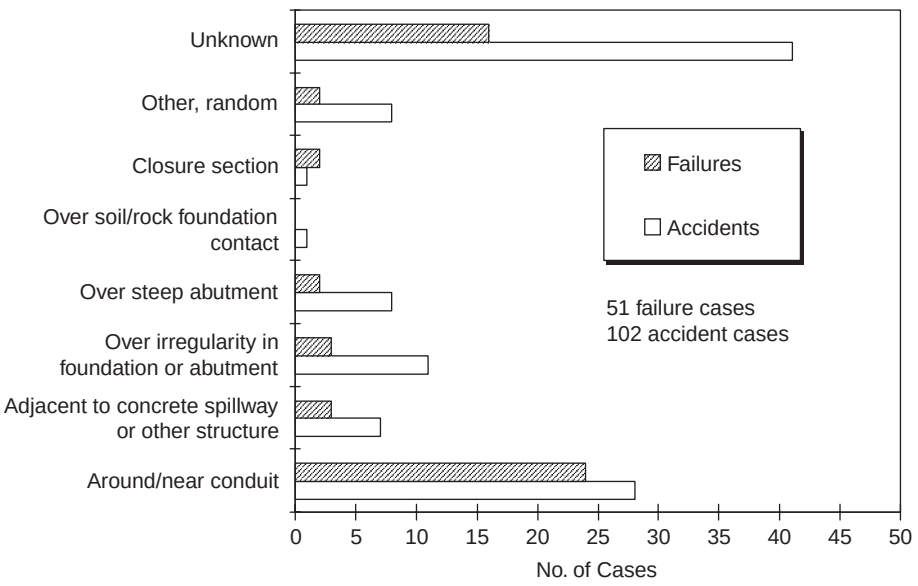


Figure 20.3 Location of internal erosion and piping failures and accidents for piping in the embankment (Foster *et al.*, 1998).

For internal erosion and piping in the foundation the reservoir water level was often at or above the historic high, but there were some failures with somewhat lower reservoir levels.

Figures 20.4 and 20.5 show observations during piping incidents in the embankment and foundation.

20.2.4 Time for development of internal erosion and piping failure of embankment dams and ease of detection

Fell *et al.* (2001, 2003) describe an approximate method for estimating the time from first concentrated leak to initial breach of the dam. The method is summarized in Section 8.9.2.

The times are for situations where failure occurs. Dams with properly designed and constructed filters, or even filters providing only some degree of protection, will usually not fail or the piping will develop more slowly, increasing the likelihood of intervention.

The times are from the first sign of a concentrated leak to first breach of the dam crest. They therefore cover the progression phase of the internal erosion and piping process. These methods for assessing the rate of development of internal erosion and piping are very approximate and should be only used as a general guide to performance. Where the consequences of failure of the dam are large, a cautious approach should be adopted in making and using these estimates.

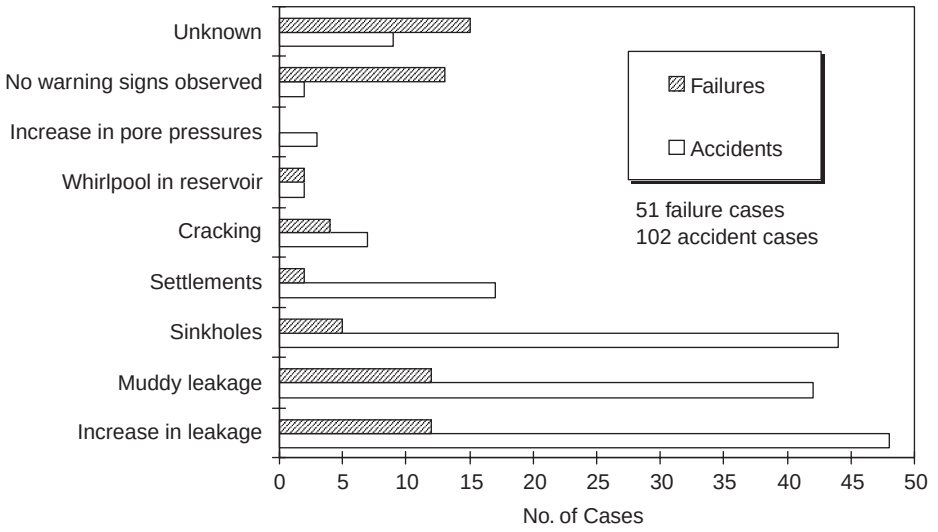


Figure 20.4 Observations during piping incidents – piping through the embankment (Foster *et al.*, 1998).

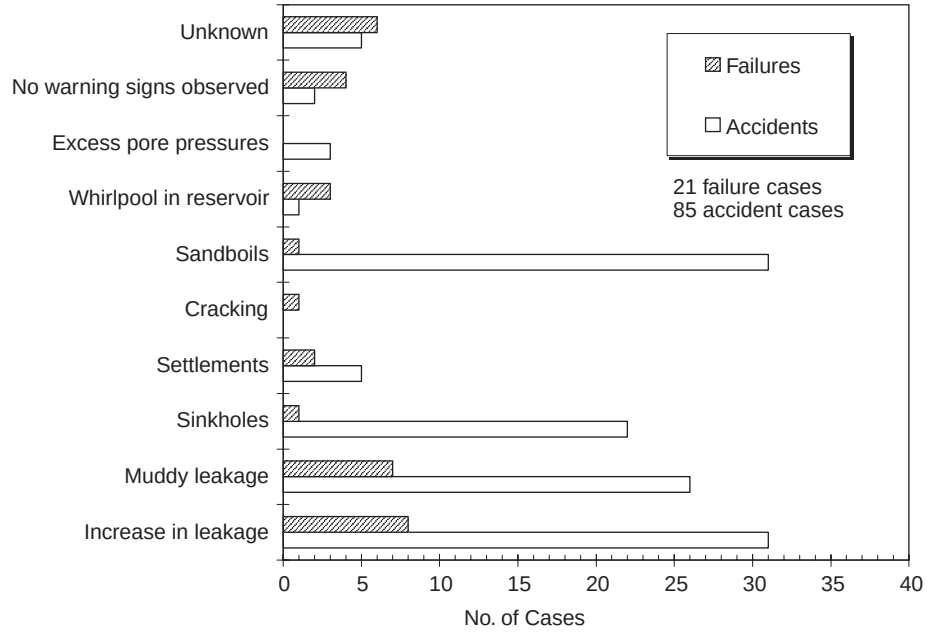


Figure 20.5 Observations during piping incidents – piping through the foundation (Foster *et al.*, 1998).

It should be recognized that the time for progression and development of a breach does not include the time for the reservoir to be emptied of water, i.e. it does not include the time for breach enlargement which is used in estimates of dam-break floods. It also does not equal the available warning time to evacuate persons at risk, since this is dependent on whether the dam is under surveillance at the time of the piping incident starts and develops and the measures set in place to advise persons downstream of an impending dam failure.

The times for initiation and continuation of erosion are dependent on the initiation mechanism, and the ability of filters, if present, to control erosion. Fell *et al.* (2002, 2003) have made a judgemental assessment of the usual rates of development and ease of detection. These are summarized in Tables 20.2 to 20.4.

In these tables the terms are defined as follows:

Rate of development: Slow = weeks or months, even years; Medium = days or weeks; Rapid = hours (>12 hours) or days; Very Rapid = <3 hours.

Ease of detection: Difficult = unlikely to be detected in most cases; Moderate = may be detected in some cases; Readily = readily detected in most cases.

Methods of detection: Visual (inspection), seepage (measurement either visual or by instruments), pore pressure (measurement); survey (survey of surface markers to determine horizontal and vertical deformation); thermal (thermal measurements within or on the downstream surface of the embankment including use of fibre optics); geophysical (resistivity and self potential).

The classifications for ease of detection assume the dam is well instrumented to measure seepage and pore pressure, readings are frequent and the dam is subject to regular inspection by experienced personnel. These classifications will be too optimistic if the dam and the area downstream are covered in vegetation, or snow covered for part of the year or if the toe of the dam is drowned by the river or another reservoir downstream.

The following should be noted:

(a) Monitoring of seepage, either by visual surveillance, or measurement, is the most common means of identifying whether internal erosion and piping have occurred.

(b) It is not common to have sufficient change in the seepage, or in other factors such as pore pressure changes or settlement, to identify conclusively that internal erosion has initiated and is continuing. It is more common to recognize when the erosion has progressed to the stage that a pipe has developed; or that there are changes in pore pressures, seepage or settlement which may be related to internal erosion, but this is not conclusive and it may reflect other factors. These changes may be a pre-cursor to a higher likelihood of internal erosion and piping, so it is important they are observed and, when they occur, the frequency of monitoring should be increased and the causes investigated.

(c) The inability to detect that internal erosion has initiated relates to the common mechanisms of initiation. For piping in embankment failures, initiation is most common in cracks, high permeability zones or hydraulic fractures in the embankment or around a conduit. These mechanisms could be expected to initiate very rapidly or rapidly once the reservoir level reaches the critical level at which erosion begins in cracks or high permeability zones or the critical level at which

Table 20.2 Rate of development and detectability of internal erosion and piping – piping through the embankment (adapted from Fell *et al.*, 2001, 2003).

Phase of Development	*Mechanism*	*Usual Rate of Development*	*Ease of Detection*	*Method of Detection*
Initiation	Global backward erosion in the core	Medium to slow, potentially rapid to very rapid at the end	Difficult (unless observed at downstream face)	Pore pressure, seepage, thermal, geophysical Visual if emerges on the downstream face.
	Concentrated leak in a crack or in a hydraulic fracture	Rapid or very rapid, medium if soil very resistant to erosion	Difficult	Seepage, visual if the crack emerges on the downstream face
	Concentrated leak in cracks or hydraulic fracture, associated with conduits or walls	Rapid or very rapid	Difficult	Seepage, visual.
	Suffusion/internal instability	Rapid for internally unstable soils. Medium to slow if soils graded so internal instability is marginal	Moderate to difficult	Pore pressure, seepage, thermal, geophysical, visual if emerges on the downstream face
Continuation	Filters satisfying no-erosion criteria	Erosion will cease		
	Filters satisfying excessive erosion criteria	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
	Filters not satisfying continuing erosion criteria, or no filters	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Progression to form a pipe, and a breach	Gross enlargement; and slope instability linked to development of a pipe.	Very rapid or rapid	Moderate to readily	Seepage, visual
Assess time using method in Section 8.9.2	Crest settlement or/and sinkhole in embankment	Usually slow to medium.	Readily	Visual, survey, seepage
	Slope instability, unravelling or sloughing	Usually slow to medium unless linked to rapid development of pipe.	Readily to Moderate	Visual, survey, seepage

hydraulic fracture initiates. However this depends on the erosion rate properties of the soil.

(d) Progression of erosion by suffusion (internal instability) is likely to be a more slowly developing process, accompanied by more gradual increases in seepage and changes in pore pressure with time. Some cases where sinkholes develop in the reservoir upstream of a dam founded on alluvial or fluvioglacial soils may be due to suffusion.

Table 20.3 Rate of development and detectability of internal erosion and piping – piping through the foundation, and from the embankment to the foundation (adapted from Fell *et al.*, 2001, 2003).

Phase of development	*Mechanism*	*Usual Rate of Development*	*Ease of Detection*	*Method of Detection*
Initiation – Foundation	Backward erosion piping with eroding soil strata exposed downstream	Rapid if critical gradient is reached	Readily if at surface. Difficult if eroding into ditch or channel	Visual, seepage, pore pressure, thermal, geophysical
	Backward erosion following "blow-out" of overlying soil strata	Rapid to very rapid	Readily to difficult	Visual, seepage, pore pressure, thermal, geophysical
	Concentrated leak erosion	Rapid or very rapid	Moderate to difficult	Visual, seepage
	Suffusion/internal instability	Slow	Moderate to difficult	Visual, seepage, pore pressure, thermal, geophysical
Initiation – embankment to foundation	Backward erosion (not applicable to plastic soils)	Slow, potentially rapid to very rapid at the end	Difficult	Pore pressure, seepage
Initiation – embankment to foundation	Contact erosion	Medium to rapid depending on soil erosion properties	Difficult	Pore pressure, seepage, survey, thermal, geophysical
Continuation	Filters satisfying no erosion criteria	Erosion will cease rapidly if all seepage is intercepted		
	Filtered exit, satisfying some or excessive erosion, or incomplete seepage interception	Filtering should become effective rapidly to medium Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
	Filtered exit, not satisfying continuing erosion criteria, or unfiltered exit	Erosion will continue rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Progression to form a pipe, and a breach	Gross enlargement in the foundation or in the embankment; and slope instability linked to development of a pipe.	Very rapid or rapid	Moderate to readily	Seepage, visual
Assess time using method in Section 8.9.2.	Crest settlement or/and sinkhole in embankment	Medium or slow Usually slow to medium.	Readily	Visual, survey, seepage
	Slope instability, (and unravelling or Sloughing for internal erosion embankment to foundation	Usually slow to medium unless linked to rapid development of pipe.	Readily Moderate to readily	Visual, survey, seepage Visual, survey, seepage

Notes:
(A) Filters are those controlling internal erosion around the conduit, or adjacent walls.
(B) Often the crack and seepage is visible on inspection, but erosion may be intermittent, or only when conduit is flowing with water, so not readily observed.
(C) The evidence seems to be that even for conduits surrounded by erodible soils, e.g. fine sand, the rate of erosion into the conduit is slow.

Table 20.4 Rate of development and detectability of internal erosion and piping – piping into and along conduits or adjacent to walls (adapted from Fell *et al.*, 2001, 2003).

Phase of Development	*Mechanism*	*Usual Rate of Development*	*Ease of Detection*	*Method of Detection*
Initiation	Concentrated leak in crack or hydraulic fracture	Rapid or very rapid	Difficult	Visual, seepage
	Erosion into open crack or joint in the conduit or wall	Slow	Moderate	Visual, seepage
Continuation Concentrated leak in crack or hydraulic fracture	Filters (A) satisfying no erosion criteria	Erosion will cease rapidly		
	Filtered exit, satisfying excessive erosion	Filtering should become effective rapidly to medium	Moderate to difficult	Seepage, pore pressure
	Filtered exit, not satisfying continuing erosion criteria, or unfiltered exit	Erosion will continue rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Continuation Erosion into open crack or joint in the conduit or wall	Crack or joint width satisfies excessive erosion criteria.	Slow (C)	Moderate to difficult (B)	Seepage, pore pressure
	Crack or joint width does not satisfy continuing erosion criteria	Slow to medium (C)	Moderate to difficult (B)	Seepage, pore pressure
Progression to form a pipe and a breach	Gross enlargement; and slope instability linked to development of a pipe	Very rapid or rapid	Moderate to readily	Seepage, visual
Assess time using method in Section 8.9.2.	Crest settlement or/and sinkhole in the embankment	Usually slow to medium.	Readily	Visual, survey, seepage
	Slope instability, unravelling or sloughing	Usually slow to medium unless linked to rapid development of pipe.	Moderate to readily	Visual, survey, seepage
	Erosion into a conduit or crack in a wall, usually only progresses towards breach by initiation of erosion along the conduit or wall, in which case factors are as for gross enlargement and instability. For cases which continue towards development of a sinkhole in a crest mode of breach, ability to hold a roof/or sinkhole is critical and the rate of development is usually slow, readily detectable by visual, survey or seepage.			

Notes:
(D) Filters are those controlling internal erosion around the conduit, or adjacent walls.
(E) Often the crack and seepage is visible on inspection, but erosion may be intermittent, or only when conduit is flowing with water, so not readily observed.
(F) The evidence seems to be that even for conduits surrounded by erodible soils, e.g. fine sand, the rate of erosion into the conduit is slow.

(e) 'Blow-out' or 'heave' in dam foundations where seepage forces create a zero effective stress condition is a situation which should be readily detected by carefully positioned and well monitored piezometers. Often these low effective stresses occur below lower permeability layers which act to confine the seepage flow and it is important that piezometers are installed to measure pore pressures in these areas. These pressures are usually directly related to reservoir levels and it is important to monitor the relationship between the pore pressures and reservoir levels. Most often these conditions will occur on first filling or at historic high reservoir water levels, but the authors are aware of cases where pressures have increased with time possibly due to suffusion in the foundation soils or blockage of drains and pressure relief wells.

(f) Failures from piping in the foundation and from the embankment to the foundation are mostly from backward erosion, or backward erosion following hydraulic fracture, 'blow-out' or 'heave'. These would not necessarily be expected to be preceded by large increases in seepage during the time the erosion is gradually working back from the downstream exit point. There will commonly be development of sand boils but if erosion is into a ditch or channel below water it may be difficult to detect. When the erosion has progressed to within a short distance of the reservoir/foundation interface, it breaks through rapidly or very rapidly.

(g) Accidents are also usually detected in the progression phase, rather than in the initiation and continuation phase. Progression ceases due for example to sealing of eroded materials on filters/transitions which satisfy excessive erosion criteria, flow limitation by an upstream dirty rockfill zone, or concrete face (on concrete faced dams), or due to collapse of the pipe. The latter is possibly a more common mechanism in foundation piping accidents than in embankments, because most embankment cores will have sufficient fines to support a roof to the pipe.

(h) Timely intervention can change a potential failure into an accident. As discussed in (b), changes in pore pressure, seepage, and settlement may be detected before internal erosion progresses too far.

(i) The ease of detection of internal erosion and piping in the foundation by visual and seepage means is more readily achieved if the area downstream of the dam is not vegetated, or is at least cleared of larger vegetation. Detection is difficult if the area downstream is densely vegetated or if the dam toe area is covered by water.

(j) Detection by pore pressure measurement is more likely if there is extensive instrumentation that is read regularly and if the erosion is widespread. However it is unlikely piezometers will detect concentrated leak erosion because for piezometers to detect the initiation of concentrated leak erosion would require at least one of them to be in the crack or poorly compacted layer. Apart from defects formed by the insertion of a piezometer the likelihood of this occurring is low.

(k) The frequency at which seepage is measured can be related to the likelihood that a dam may fail or experience an accident by internal erosion and piping, the consequences of failure and the likely time for internal erosion to progress to form a pipe and the dam breach. It might also be related to the reservoir level, with enhanced measurements at times of high reservoir level. There is strong evidence that failures and accidents from internal erosion and piping occur above or near historic high reservoir levels. At such times the dam is also at greatest risk from

a slope instability point of view. For dams with known deficiencies in internal erosion and piping or instability, there should be a significantly increased level of inspection and monitoring under these reservoir conditions. Indeed, it would be good practice to increase the surveillance for all embankment dams.

(l) The seepage monitoring system needs to be capable of being calibrated to separate out the effects of rainfall or snowmelt e.g. by prior observation and monitoring and coupling seepage measurements to rainfall and snowmelt measurements at the dam. It should however be recognised that there are many situations where it will be unlikely that seepage will be detected, e.g. at night when visual surveillance is being relied upon; if the toe of the dam is submerged; if seepage occurs high on the dam abutments and bypass measuring weirs or in winter when the dam is covered by snow.

(m) There are clearly many dams which may have progression and breach times of the order of hours. These dams particularly include some older dams which have no filter or transitions, dams without downstream rockfill zones to stop or slow the erosion process, and/or those dams on erodible foundations without well designed and constructed cutoffs or filters to intercept the foundation seepage. For these dams, an effective seepage monitoring program would require virtually continuous monitoring. Daily, or even twice daily inspections, or measurements may be inadequate.

In view of these factors it is the authors' opinion that:

(a) It is questionable whether it is good practice to install piezometers in the core of new dams because they are unlikely to be useful in detecting internal erosion, and the methods used to install the tubes or leads to the piezometers may result in defects in the core. At a Workshop on internal erosion and piping in USA attended by many experienced dam engineers the majority held this view. Some however still favour having piezometers. If they are to be installed there must be much attention paid to the design details of the installation of the piezometers, and in fact all embedded instrumentation in the core, if the deliberate creation of defects in the core is to be avoided. Some past installation practices are clearly not good from this point of view, so the designers of new dams must think carefully about the installation details and not blindly adopt ones that have been used before.

(b) Except in circumstances where there are concerns about a dam, piezometers should not be installed in existing dams because the process of installing the piezometer in boreholes may cause cracking by hydraulic fracture from the drilling process or backfilling with grout. If piezometers are installed in these dams dry drilling methods such as solid or hollow flight auger or sonic drilling with no drilling water, mud or air must be adopted, and grouting of the hole carried out in short lengths (say 3 m to 5 m) with time for the grout to set between stages so it does not cause hydraulic fracture.

(c) It is good practice to install piezometers in the foundation of dams on cohesionless soils where there is no positive cut-off through the cohesionless soil. These should be installed near the downstream toe and positioned to be at the top of the cohesionless layers of soil where these are overlain by lower permeability strata because this is where heave is most likely to occur.

20.2.5 The ability of monitoring to detect slope instability

When an embankment is in a state of marginal stability it is likely to show signs of this with increased rates of settlement or deformation compared to the normal on-going deformations which occur in dams. Examples of these are Cairn Curran dam (Swindon *et al.*, 2004), Hume No 1 embankment (Cooper *et al.*, 1997) and Eppalock Dam (Davidson *et al.*, 2001). For Cairn Curran the first sign of marginal stability was detected when it was noted that the kerb on the crest of the embankment near the spillway had settled and moved upstream.

In view of this, monitoring of deformations on a regular basis is likely to detect slope instability in many cases. However if the soils are markedly strain weakening failure may occur before the deformations are detected. For instability through the foundation it is important to have survey marks at and just beyond the toe of the embankment to detect the movements there.

Piezometers are likely to be useful in detecting high pore pressures which may be a precursor to slope instability. This is particularly so for situations where there are high foundation pore pressures affecting stability. However it is very unlikely the piezometers will be positioned in the actual failure surface shear zone so they may not give a true indicator of pore pressures and will not measure the positive pore pressures developed in shearing of contractive soils. It is the authors' opinion that monitoring of surface deformations is an important part of dam safety management. There should be survey marks on the crest, and upstream and downstream slopes. It is most important to have survey marks where embankments abut spillway and other walls because experience is that is where slope instability has developed. Often existing marks are too far from the junction. Marks are needed as close as 5 metres from the junction.

Annual survey is sufficient after first filling. Longer gaps between surveys are not sufficient to detect trends. Survey should be of horizontal and vertical displacements so displacement vectors can be determined.

If a dam shows signs of excessive or accelerating deformations the interval between surveys should be reduced and possibly additional survey points added to better define the extent of the deformations. In some situations it may be necessary to install piezometers and borehole inclinometers to better determine the depth and nature of the deformations. If this is done care should be taken to avoid hydraulic fracture as described above.

20.3 WHAT INSPECTIONS AND MONITORING IS REQUIRED?

20.3.1 General principles

As discussed in ICOLD (1989) and ANCOLD (2003), the inspection and monitoring appropriate for a dam depends on a number of factors:

(a) The consequences of failure. Dams with a higher consequence of failure in terms of potential lives lost, economic and environmental damage, require a higher level of inspection and monitoring.
(b) The type of dam. For example, dams which have no filters or drainage zones, may require greater inspection and monitoring than a well designed and constructed dam with filters, assuming the same consequences of failure.

(c) The type of dam foundation. A dam on permeable soil foundations would usually require more inspection and monitoring than the same dam on a rock foundation.
(d) The size of dam. This is usually linked to (a) and (b), but in general larger dams have a higher level of inspection and monitoring than small dams.
(e) Known deficiencies or deterioration of the dam. If there are identified safety issues, such as marginal slope stability or larger than normal seepage or pore pressures, there will usually be a need for enhanced inspection and monitoring until the deficiencies are rectified.
(f) Identified potential failure modes.
(g) The age of the dam. There is a need for enhanced inspection and monitoring during first filling of the reservoir, because it is known that many failures and accidents occur in this period.

To this the authors would add:

(h) The reservoir level, compared to historic high levels. There is strong evidence that failures and accidents from internal erosion and piping occur above or near historic high reservoir levels. At such times the dam is also at greatest risk from a slope instability point of view. For dams with known deficiencies in internal erosion and piping, there should be a significantly increased level of inspection and monitoring under these reservoir conditions. As an over-riding rule, it would be wise to do this increased monitoring for all embankment dams at times when the reservoir is near its historical high water level.
(i) The reservoir level, compared to historic low levels. Many dams with marginal upstream slope stability, or subject to internal deformations due to poorly compacted rockfill, experience abnormal settlements and deformations when the reservoir is at or below historic low levels and inspection for cracking and survey of deformations should be planned at that time and again when the reservoir fills.
(j) Post earthquake. Inspection and monitoring should be done after significant earthquake loading on the dam. Such increased inspection and surveillance regime will probably have to be continued for some time, even beyond the time when the reservoir level has been lowered far enough for the Owner to have confidence about the dam's immediate level of stability and, in the case of embankment dams, the risk of post-earthquake piping developing.

The inspection and monitoring required for a dam should be assessed by experienced dam engineers, including the designers (for new dams), and the engineer(s) responsible for surveillance operation and maintenance of the dam. An experienced engineering geologist, preferably with knowledge of the site, must be included in this team whenever questions about the foundations have been or might be raised.

There will often be compromises to be made, offsetting the costs of instrumentation, readings and surveillance against what is desirable. For older dams, when instruments fail, there are often difficult decisions to make on whether they should be replaced, taking into account the risks in replacing them e.g. the risk of damaging the dam core by drilling to replace piezometers must be weighed against the benefits gained in having the monitoring data.

Table 20.5 Number of applications of detection methods used to detect deterioration in embankment dams (ICOLD, 1983a).

Detection methods	*Deteriorations affecting foundations*	*Deteriorations affecting dam body*	*Deteriorations affecting foundations and dam body*	*Total*	
				No.	*%*
Direct observation	92	250	59	401	63
Water flow measurements	37	30	17	84	13
Phreatic level measurements	8	2	2	12	2
Uplift measurements	1	–	–	1	<1
Pore pressure measurements	17	8	5	30	5
Turbidity measurements	3	1	2	6	1
Chemical analysis of water	3	1	2	4	1
Seepage path investigations	1	5	2	8	1
Horizontal displacement measurement	3	25	2	30	5
Vertical displacement measurements	5	27	7	39	6
Strain measurements	–	1	1	2	<1
Rainfall measurements	–	1	–	3	<1
Sounding investigation	2	1	–	3	<1
Design revision (new criteria)	1	3	–	4	<1
Not available	3	3	1	6	1
Total of deterioration cases	109	260	63		

Table 20.5 summarises the detection methods which led to the identification of deterioration of embankment dams in the ICOLD (1983a) study.

The authors believe that the most important monitoring and surveillance measures are:

- Inspection by trained observers.
- Seepage flow measurement.
- Vertical and horizontal displacement measurement by direct survey of surface points.
- Pore pressure measurement.
- Measurement of reservoir water level and rainfall (as these link to the other factors).

Other items which may be monitored include:

- Internal deformations, using internal settlement gauges, extensometers, or borehole inclinometers.
- Seepage water temperature.
- Seepage water chemistry and pH.
- Seepage water turbidity measurement.
- Seismic ground motion.

The authors also believe strongly that the data from the instrumentation and the inspection records must be reviewed by an experienced dam engineer as soon as they are received from the field inspection team. The whole exercise becomes almost pointless

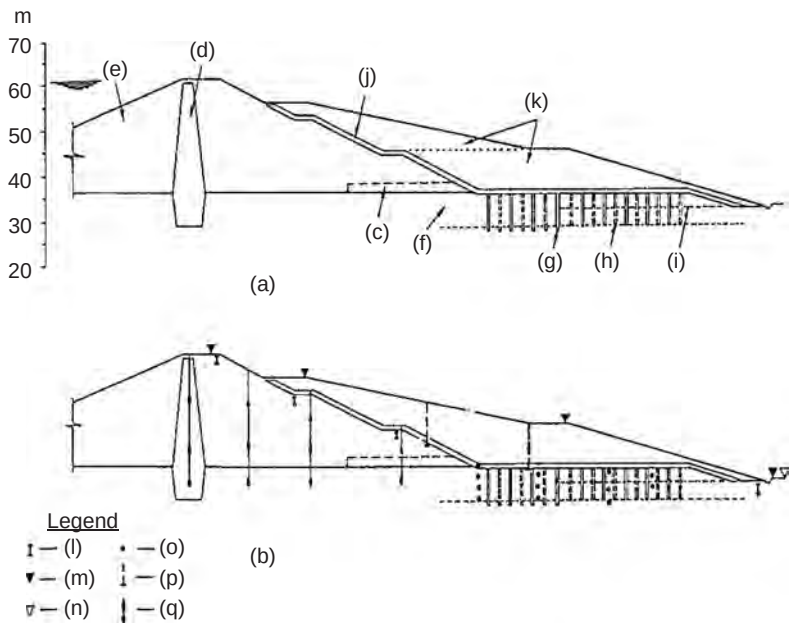


Figure 20.6 Monitoring for Prospect earthfill dam (ICOLD, 1989). (a) Cross section showing old dam and fill buttressing. (b) Cross section showing monitoring systems. (c) Rubble filled drainage tunnel. (d) Puddle clay core. (e) Clay shoulders. (f) Alluvial clay foundation. (g) Filter drainage cutoff trench along downstream foundation extending to rock. (h) Vertical sand drains. (i) Lateral drains. (j) Filter zone. (k) Stabilizing fill material (shale) constructed in two stages. (l) Inclinometers. (m) Surface settlement points. (n) Seepage collection and measurement. (o) Pneumatic and electric piezometers. (p) Foundation settlement installations. (q) Open standpipe piezometers.

if nobody with the necessary knowledge looks at this field data. We are aware of one case where the collected data was reviewed every 5 years, which makes a nonsense of the inspection and surveillance of that particular dam.

20.3.2 Some examples of well instrumented embankment dams

Figures 20.6 to 20.9 show examples of quite intensive monitoring for Prospect Dam (a 100 year old 35 m high puddle core dam, constructed of erodible soil with no filters in the original construction); Cethana Dam, a 110 m high CFRD; Trinity Dam (150 m high earth and rockfill) and Svartevann Dam (129 m high earth and rockfill) (all from ICOLD 1989).

Table 20.6 is based on ANCOLD (2003) guide for the frequency of monitoring for embankment dams based on the consequences of failure category. These would be appropriate for a dam without known deficiencies, and after first filling.

During construction and first filling, more frequent readings of monitoring instruments would be appropriate.

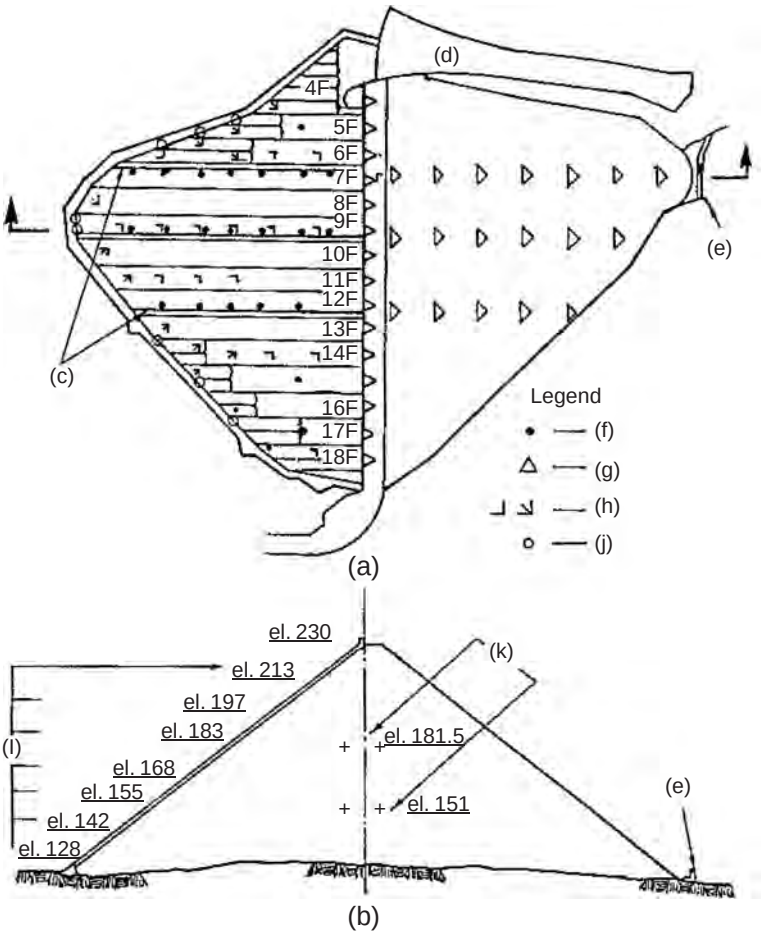


Figure 20.7 Monitoring for Cethana concrete face rockfill dam (ICOLD, 1989). (a) Plan view. (b) Cross section. (c) Inclinometer pipes. (d) Spillway. (e) Leakage weir. (f) Anchor points for wires. (g) Survey targets. (h) Strain gauges. (j) Perimetric joint meters. (k) Hydrostatic settlement cell. (l) Installation level of strain gauges.

Table 20.7 gives an example for a large embankment dam, showing how the frequency of readings changes as the satisfactory performance of the dam is confirmed.

20.3.3 Dam safety inspections

Dam safety inspections are the most important part of any dam surveillance program. As pointed out by ANCOLD (2003), inspections should be carried out by experienced people, trained to recognize deficiencies in dams. Inspections requiring technical evaluation, should generally be carried out by a dams engineer and other specialists. The day-to-day inspections which would detect many deficiencies are best done by trained operations personnel. Dam owners should ensure that all operations personnel are

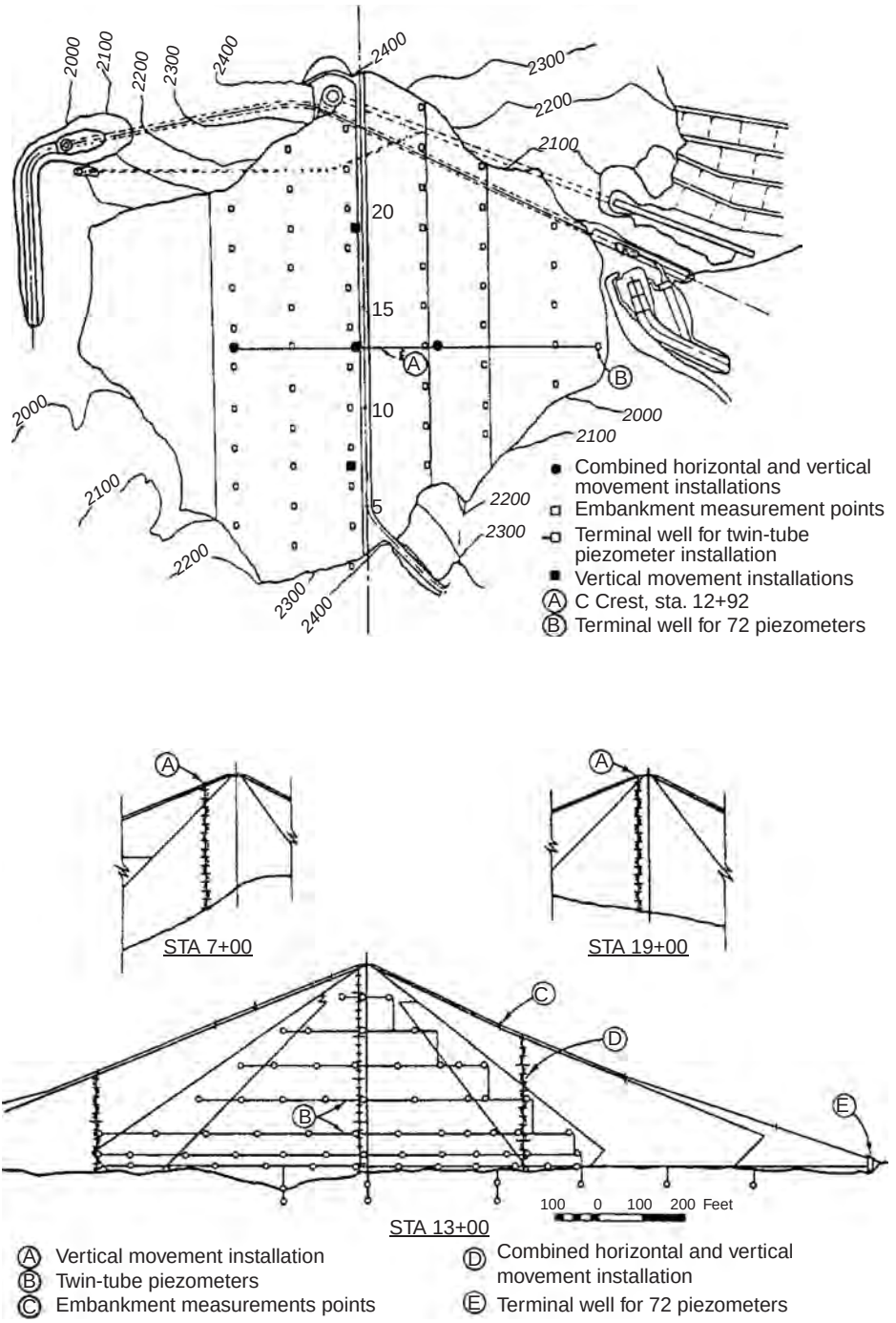


Figure 20.8 Monitoring for Trinity earth and rockfill dam (ICOLD, 1989).

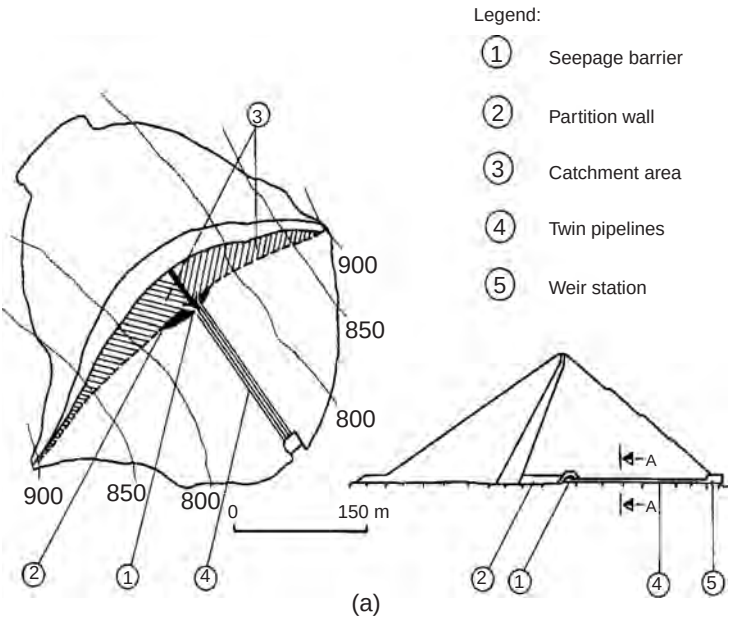
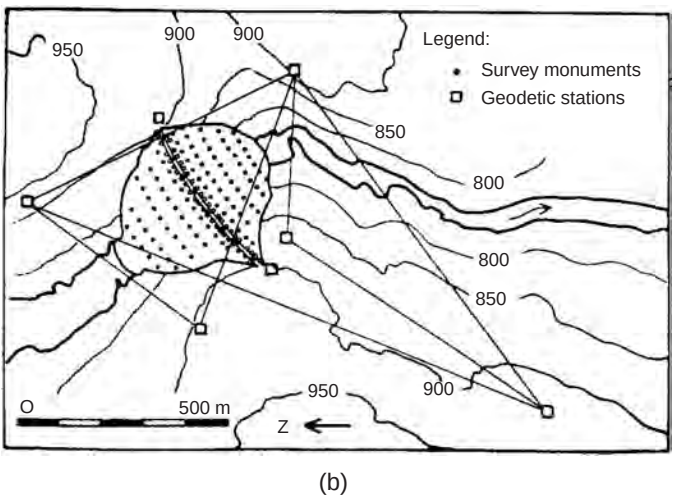
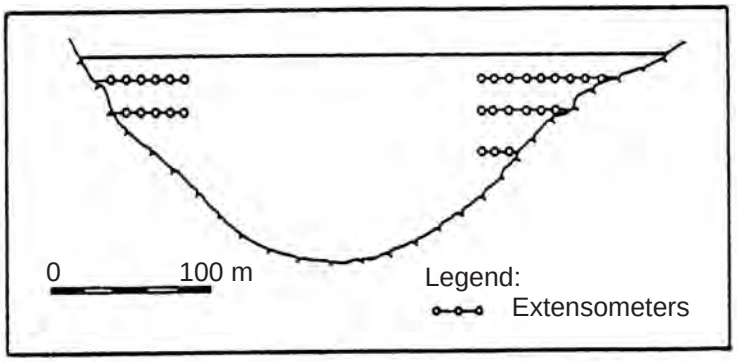


Figure 20.9 Monitoring for Svartevann earth and rockfill dam: (a) Leakage measurement; (b) Surface displacement; (c) and (d) Internal displacement and strains; (e) Pore pressures; (f) Earth pressure (ICOLD 1989).

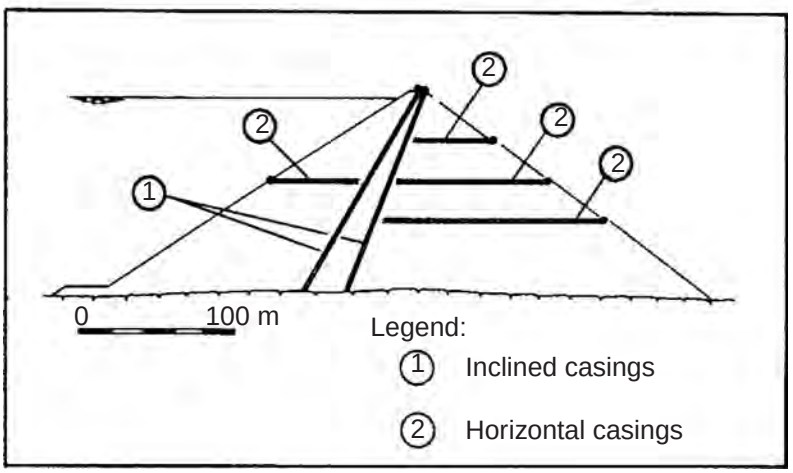


suitably trained and are aware of the consequences of failure of the dam and of the deficiencies that have been found in similar dams.

Ideally dam owners should arrange for these inspectors to be given the chance to spend time with inspectors from other like-minded dam owners to see what issues have arisen at other dams.



(c)



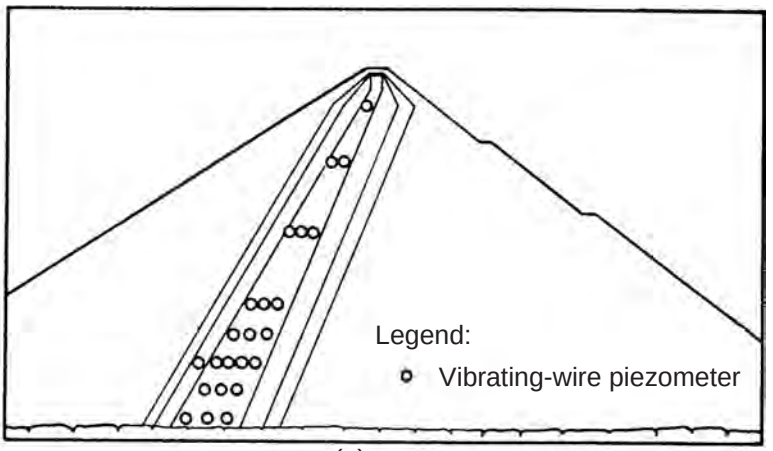
(d)

Figure 20.9 Continued.

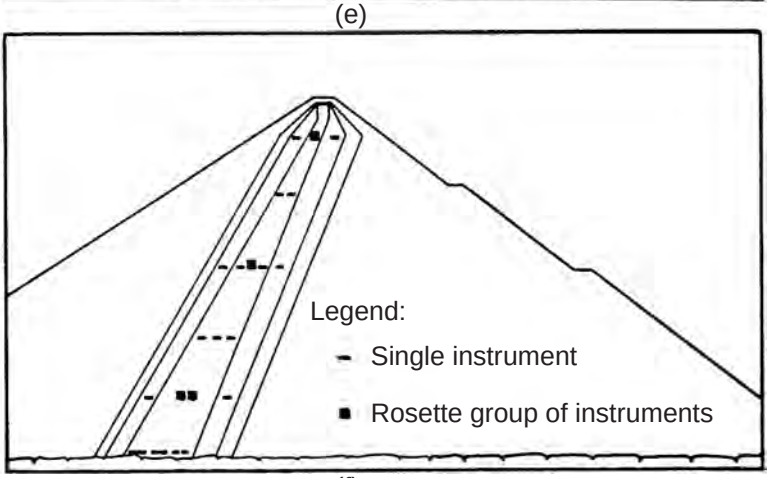
ANCOLD (2003) recommend five levels of inspection which are detailed in Table 20.8.

For dams in sound condition with no deficiencies, ANCOLD (2003) recommend the frequency of inspection shown in Table 20.9. More frequent inspections may be necessary where a dam is known to have a deficiency. The authors would add that more frequent inspections should be taken under historic high or as the reservoir is re-filling after historic low reservoir conditions.

The frequency of routine dam safety and routine visual inspections should be increased during first filling. For example, for a high or extreme consequence of failure dam which fills over a few months, there should be daily routine visual inspections, and say weekly or fortnightly routine dam safety inspections by engineers familiar with the design of the dam and its expected performance. The inspection should be carried



(e)



(f)

Figure 20.9 Continued.

out to a checklist. An example of such a checklist is given in Table 20.10 the checklist is for an earth dam, but could be modified for other types of dams. Seepage measurement would normally be included on the same report sheet. USBR (1983), FEMA (1987), FERC (2005), ICOLD (2014) and ANCOLD (2003) have more detailed checklists.

20.4 HOW IS THE MONITORING DONE?

20.4.1 General principles

Some good general principles on dam monitoring are given by the French National Committee in ICOLD (1989):

a. Dams are structures with a long life span. *Reliable* instruments with a similar life span are necessary, and they must be accessible, verifiable and replaceable.

Table 20.6 Guide for 'in service' embankment dam monitoring frequencies[1] (adapted from ANCOLD, 2003).

	Consequences of failure category				
Monitoring	*Very low*	*Low*	*Significant*	*High*	*Extreme*
Rainfall	Monthly[5]	Monthly	Daily to twice weekly (TC)[2]	Daily to tri weekly (TR)[2]	Daily (TR)[2]
Storage level	Monthly[5]	Monthly	Twice weekly to weekly (TC)[2]	Daily to tri weekly (TR)[2]	Daily (TR)[2]
Seepage	Monthly[5]	Monthly	Twice weekly to weekly (TC)[2]	Daily to tri weekly (TR)[2]	Daily (TR)[2]
Chemical analysis and pH of seepage			Consider	Yearly	Yearly
Pore pressure[3]		Consider	3-monthly	Monthly to 3-monthly	Monthly
Surface movement, control[4]		Consider	Consider	5-yearly to 10-yearly	5-yearly
Surface movement, normal			Consider	2-yearly	Yearly
Internal displacement				2-yearly	Yearly
Seismological[3]				Consider (TC)	Consider (TR)

Notes:
1. These frequencies may need to be varied according to the conditions at, and the type and size of dam, and applies to instrumentation already installed at the dam.
2. The frequencies quoted assume manual reading of the instrumentation. Where automated readings are available more frequent reading would be appropriate. (TR – telemetry recommended; TC – telemetry to be considered).
3. The frequency of reading and location of the pore pressure instruments and seismic instruments need to be at the discretion of the dams engineer.
4. A control survey uses monuments that are remote from the dam site to check the location of the survey monuments at the dam site.
5. The frequencies listed for very low consequence of failure category dams are suggestions, the dam owner should determine appropriate monitoring.
6. Applicable for concrete dams and construction on potentially soluble materials or where there is concern about seepage in the dam or its foundations.

b. They must be *sensitive* to give warning of sudden changes or trends that are very small but may be significant indicators of distress.
c. They should be *simple* so that frequent readings can be made quickly by site or operating staff without the need for specialists; in this way, the dam is more or less permanently monitored, and the records are available for statistical analysis.
d. In recent decades far more attention has been focused on deformations and hydraulic behaviour in the *foundations*.
e. V*isual inspection* and surroundings, by people who know the dam, is very important because even the best instruments will not find fissures, leakages, dam spots or their growth, etc.
f. The choice of parameters to be measured and the positioning of instruments cannot always be optimal at the design stage. The number of instruments sometimes amazes but is justified by the need for effective monitoring of a so far unfamiliar structure. Optimisation of the system can be made by abandoning some instruments or installing others as the real behaviour becomes better understood.

Table 20.7 Monitoring for Revelstoke Dam (Canadian National Committee in ICOLD, 1989).

	Instruments									
Stage	*Core piezometer*	*Foundation and shell piezometer*	*Vertical movement gauge*	*Horizontal movement gauge*	*Horizontal strain gauge*	*Surface monument*	*Earth Pressure cell*	*Weir and well*	*Strong motion accelerograph*	*Visual inspection*
During construction	Frequently by field staff				1/month	1/month	1/month	1/month	Continuous	1/month
Reservoir filling	Every 2 days	Every 2 days	1/week	2/month	2/month	1/month	2/month	Every 2 days	Continuous	1/day
After the first reservoir filling										
First 6 months	1/week	1/week	1/month	1/month	1/month	1/month	1/month	1/month	Continuous	1/day
6 month to 1½ year	2/month	2/month	4/year	4/year	4/year	4/year	4/year	2/month	Continuous	1/week
1½ year to 2½ year	2/month	2/month	4/year	4/year	4/year	4/year	4/year	2/month	Continuous	1/week
2½ year to 6½ year	6/year	6/year	2/year	2/year	2/year	2/year	6/year	6/year	Continuous	2/month
Subsequent years	2/year	2/year	1/year	1/year	1/year	1/year	1/year	2/year	Continuous	1/month

Table 20.8 Dam safety inspections as recommended by ANCOLD (2003).

Type of Inspection	*Personnel*	*Purpose*
Comprehensive	Dams engineer and specialists[1] (where relevant)	The identification of deficiencies by a thorough onsite inspection; by evaluating surveillance data; and by applying current criteria and prevailing knowledge. Equipment should be test operated to identify deficiencies. For a safety review consider: – Draining of outlet works for internal inspection – Diver inspection of submerged structures.
Intermediate	Dams engineer	The identification of deficiencies by visual examination of the dam and review of surveillance data against prevailing knowledge with recommendations for corrective actions. Equipment is inspected but not necessarily operated.
Routine dam safety	Inspector	The identification and report of deficiencies, by structured observation of the dam and surrounds, by an inspector, other than the operator, with recommendations for corrective actions.
Routine visual	Operations personnel	The identification and report of deficiencies by visual observation of the dam by operating personnel as part of their duties at the dam.
Special/ emergency	Dams engineer and specialists[1]	The examination of a particular feature of a dam for some special reason (eg. after earthquakes, heavy floods, rapid drawdown, emergency situation) to determine the need for pre-emptive or corrective actions.

Note:
1. Examples of specialists include mechanical and electrical engineers, to inspect outlet works, spillway gates and automated systems, and corrosion engineers).

g. Although modern information processing can help with huge masses of raw data, experience has shown that it is a good policy to read often a limited number of key instruments very well situated in the dam.
h. The first filling of a reservoir is a stage in its life that is both important and delicate. It is in fact the proof that a dam can fulfill its design functions. Of course, measurements must start earlier so that the structure's initial state is known and some instruments are read during construction so that the reaction of dam and foundation to loading are known.
i. Continuous reading of some instruments is conceivable during first filling to obtain a large amount of information during this fairly short period. During normal operation the raw data is processed by computer to give useful information. However complete automation – the automatic transmission of the state of safety of a dam and possibly of danger signs – is not used by EDF (Electricité de France). The delicate task of fine interpretation belongs to the engineer.

The authors would add:

a. The use of instruments that can be easily rechecked if readings differ significantly from previous readings or are well outside the expected range.

Table 20.9 Frequency of inspection for dams with no known deficiency (adapted from ANCOLD, 2003).

Consequence of Failure[3]	*Inspection Type*				
	Comprehensive	*Intermediate*	*Routine Dam Safety*	*Routine Visual*	*Special*
Extreme	On first filling then 5 yearly	Annual	Monthly	Daily[1]	As required
High A, B, C	On first filling then 5 yearly	Annual	Monthly	Daily[1] to tri-weekly	As required
Significant	On first filling then 5 yearly	Annual to 2-Yearly	3-Monthly	Twice weekly to weekly	As required
Low		On first filling, then 5 yearly		Monthly	As required
Very low		Dam owner's responsibility[2]		Dam owner's responsibility[2]	As required

Notes:
1. Dam owners may undertake a review to determine if a reduced or increased frequency of inspection is acceptable. The review should be carried out by a dams engineer and take into account such matters as Regulator requirements, dam consequences of failure and risk, type and size of dam, dam failure modes and monitoring arrangements.
2. Monthly routine visual inspections and 5 yearly intermediate inspections with test operation of equipment and review of consequences of failure category are suggested.
3. See Section 8.2.3 for definitions of the consequences of failure.

Table 20.10 Checklist of conditions to be noted on visual inspection (adapted from ANCOLD, 1976).

(a) Vegetation on dam and within 15 metres beyond toe of dam:
- Overgrowth: Requiring cutting for dam surveillance, requiring weed control for dam surveillance, indicating seepage or excessive seepage or moisture.
- Wet terrain vegetation: Watch for boils, sand cones, deltas, etc., changes with the season, and pond level changes.
- Incomplete – Requiring repair: Poor growth, destroyed by erosion.

(b) Foundations:
- Drainage ditches clogged with vegetation.
- Dam areas, moisture on dry days.
- Flowing water: quantity, location, clarity.
- Boils.
- Silt accumulations, deltas, cones.

(c) Embankment:
- Crest: Cracking, subsidence.
- Upstream face: Cracking bulging, surface erosion, gullying, wave erosion.
- Downstream face: Cracking, subsidence, bulging, erosion gullies; moisture on dry days; damp areas, seeps.
- Berms and within 15 metres beyond toe of dam: Erosion, gullies, damp areas, seeps.

(d) Spillways and outlet works:
- Reservoir level.
- Discharge tunnel or conduit condition, seeps, cracks.
- Seepage or damp areas around conduits.
- Erosion around or walls below conduit.
- Boils in the vicinity of conduit.
- Spillway slabs for uplift, subsidence, cracking.

(e) Areas of previous repair:
- Effectiveness of repair.
- Progression of trouble into new area.

b. If a reading does not change for some time that is no reason to stop reading that instrument.
c. Results should be quickly recorded within the owner's storage system. Wherever possible the immediately gathered data should be presented in simple, easily understood graphical plots and, as noted earlier, the data should be reviewed by an experienced dam engineer and not left for 5 years or so until the next main review is planned.

20.4.2 Seepage flow measurement and observation

Seepage data is one of the best indicators of a dam's performance. By observing the location, quantity and quality of seepage emerging from the dam embankment and its foundation, and particularly the changes which occur, one can get early warning of problems which may be developing, particularly in the important problem areas of internal erosion in the dam and its foundation and in increased pore pressures.

It is emphasized that routine observation of where seepage is emerging can be as useful a guide as the actual measurement. However, as indicated by most of the reporting countries in ICOLD (1989) and ANCOLD (1983), it is usual to measure the quantity of seepage.

As shown in Figure 20.10, it is preferable to collect the seepage close to the downstream toe of the impervious zone, and to isolate areas from each other so the readings are not influenced excessively by flow through rockfill zones and runoff from abutments.

However, as pointed out by ANCOLD (1983), for smaller dams it is acceptable to measure total seepage downstream of the toe of the dam and for larger dams, it is often impractical to have a system such as that shown in Figure 20.10. The authors know of cases where drainage trenches to subdivide the foundation into collection areas under rockfill were filled rapidly with fines from the rockfill, making the system ineffective. In other instances the seepage from individual areas simply by-passes the collector drains – the cost to seal below the drain is far too high. Where seepage is collected downstream, the influence of rainfall runoff on the readings has to be determined and separated from the base seepage flow.

Measurement of seepage may be made by:

– V-notch or similar measuring weirs. This may include continuous recording and telemetering of data for more important dams. Level switches may be provided to give an independent alarm.
– Timed discharge into measuring vessels.
– Visual inspection where flow rates are very small.

It is often impractical to collect and measure all seepage, particularly for dams on alluvial foundations. In these cases, installation and monitoring of piezometers in the foundations under and downstream of the dam can give information on changing conditions, which might indicate a problem is developing. Problems also are experienced where the toe of the dam is flooded by the tailwater from a hydropower station or irrigation outlet. In these cases collecting and measuring seepage in the body of the dam, as shown in Figure 20.10, is desirable but is seldom done in practice.

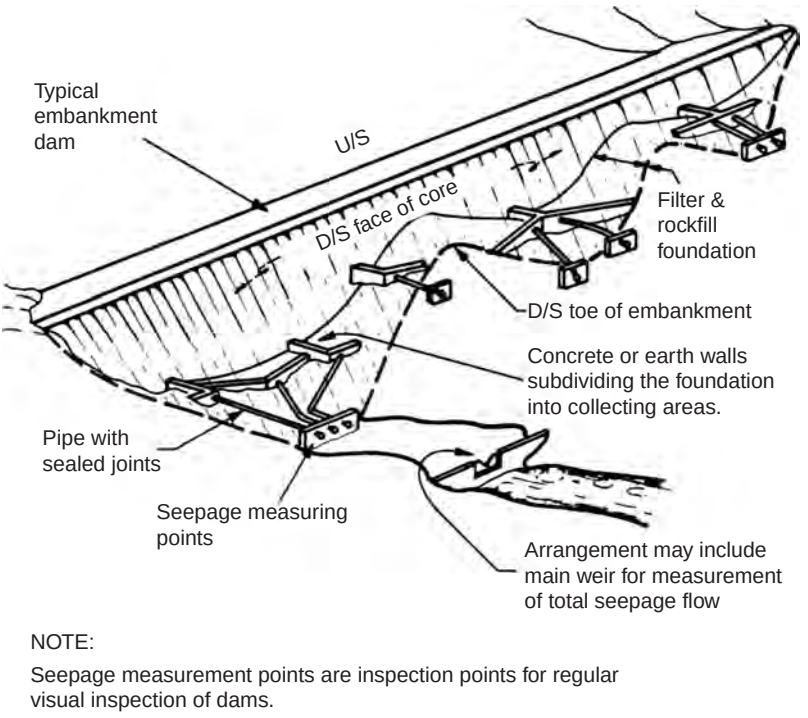


Figure 20.10 Seepage collection and measuring system for an earth and rockfill dam (ANCOLD, 1983).

Chemical analysis of seepage can be a useful guide to the source of the seepage water, e.g.:

- A comparison of ions in the reservoir water and seepage may indicate leaching of cement from grout curtains, or oxidation of sulphides within the foundation or within the embankment materials.
- Biological analysis can indicate the source relative to depth in the reservoir,
- The age of water as determined by analysis of tritium can indicate its sources as rainwater or groundwater.

More importantly, routine inspection of seepage discharge should be made noting any discolouration of the water which may indicate piping of the embankment or the foundation. Any inexplicable increase in suspended solids, particularly during first filling, needs to be treated with some urgency as piping can develop rapidly. At Eucumbene Dam in the Snowy Mountains Scheme, NSW, Australia, the turbidity is measured remotely, along with the continuous reading of the seepage.

Seepage measurements can be augmented by having some carefully placed groundwater observation holes on the downstream abutment. These holes can pick up gradually changing groundwater in the abutments and possibly help explain the influence of groundwater from hillsides above the dam.

As discussed in Section 20.4.8 there are a number of geophysical and electrical methods which are now being used to detect seepage in some dams and levees. The authors see value in these in some circumstances but do not see them replacing more conventional seepage measurement and regular inspections to detect seepage.

20.4.3 Surface displacements

Regular accurate survey of displacements of the surface of the dam embankment can be a most useful guide to performance.

Such measurements can be useful as a check on design assumptions, e.g. deformations of a CFRD, and as an indication of developing problems, including marginal slope stability and internal deformations due to softening or internal erosion and piping in the embankment or the foundation.

Figure 20.11 shows design of surface settlement points suitable for dams.

It is useful to extend the survey to surrounding areas, particularly if they are affected by slope instability.

In almost all cases, horizontal as well as vertical displacements should be measured, so that movement vectors can be determined. Such vectors can often give a good indication of the mechanism causing the displacement.

It is important to consider carefully where the survey markers are positioned, because quite different movements occur in the upstream and downstream slopes, and between the dam core and adjacent filter and rockfill zones.

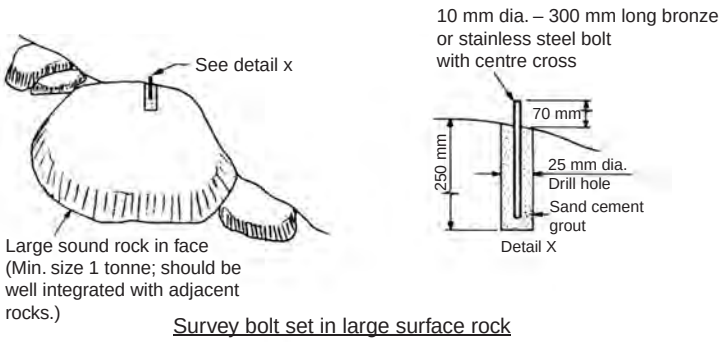
To monitor movements properly, survey markers should be positioned centrally over the earthfill core, on the upstream and downstream edges of the crest and at several levels on the upstream and downstream slopes. Survey markers (at least those on the crest of the dam) should be positioned over the whole length of the dam, not just one or two cross sections, because one of their most valuable outcomes is to detect local larger settlements. Some survey markers should always be positioned where embankments abut spillway concrete walls, because this is where local seepage, softening and abnormal deformation is often a guide to developing problems.

For monitoring of soil landslides or earthfill dams, it will be necessary to embed the survey points below the depth of shrink and swell due to seasonal moisture change. This may involve a system as shown in Figure 20.12, with the base at 2 m to 4 m below ground surface and the pillar isolated from the upper soil by a casing.

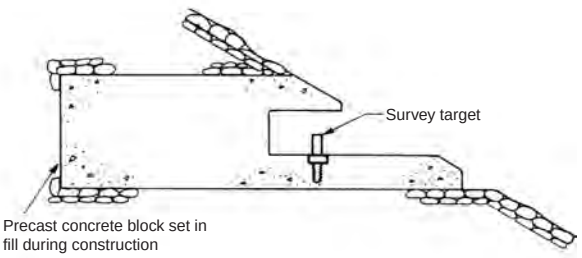
ICOLD (1988b) indicate that survey of displacements is falling out of favour. The authors' opinion is that this is unfortunate, and seemingly unnecessary, given the availability of quick accurate electronic survey methods. Our experience is that survey data has been very valuable in early identification of problems and has the great advantage of being readily checked is some results appear inconsistent.

ICOLD (2014) summarize some new methods for survey monitoring of dams. These include:

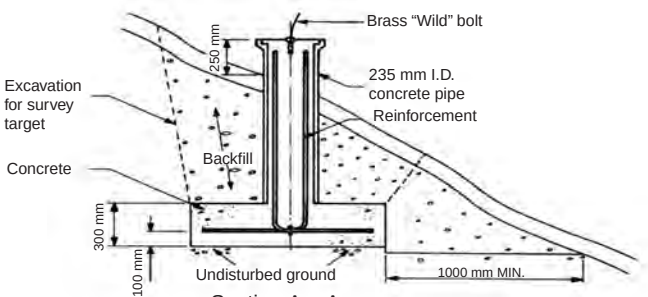
(a) Global Navigation Satellite System (GNSS). This has been used for dams in Portugal, USA and Japan for monitoring positions of monitoring points to an accuracy of 3 mm relative to other monitoring positions.
(b) Satellite survey by Synthetic Aperture Radar (SAR) and Permanent Scatters (PS) which is used to give a high resolution digital three dimensional survey to a



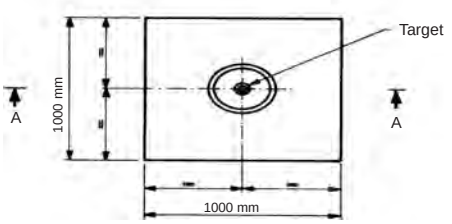
Survey bolt set in large surface rock



"Wild" bolt / precast concrete block



Section A – A



Plan

Abutment survey target

Figure 20.11 Surface settlement points for dams (ANCOLD, 1983).

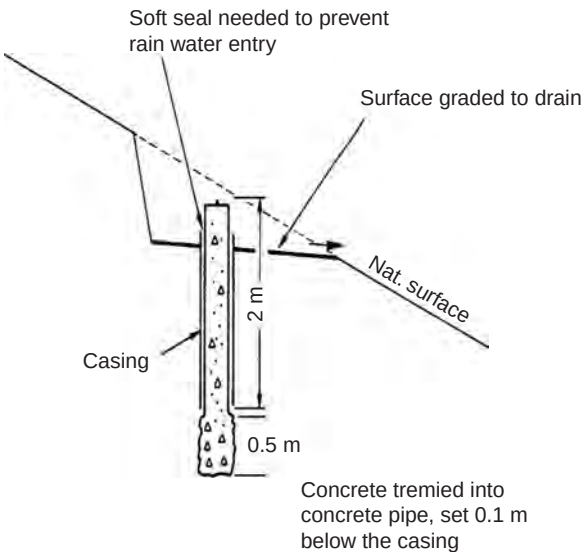


Figure 20.12 Survey point in soils susceptible to shrink and swell.

precision of ±1 mm vertical displacements at intervals of 35 days. Historic data of up to 15 years is available. There are limitations for survey of the area surrounding a dam as the method requires specific target surfaces. ICOLD (2014) indicate that measurements obtained can be as accurate as those obtained from traditional precision monitoring systems such as total station.

(c) Ground survey by SAR uses ground station located in front of the dam or reservoir slopes. Theoretically the precision is ±0.1 mm. The advantages include that it is not necessary to access the area being surveyed, surveys can be made at night and displacement maps for the whole structure can be obtained. Measurements can only be made for line of sight, and may be affected by some meteorological and light conditions. ICOLD (2014) give an example where the precision was ±0.4 mm.

20.4.4 Pore pressures

The measurement of pore pressures in the embankment and foundations of a dam can give vital quantitative information for use in assessing stability, potential 'heave' conditions in foundations and for identifying unusual seepage pressure, which may be a precursor to internal erosion and piping or slope instability provided there is adequate coverage along the dam.

The following discussion gives an overview on why and where pore pressures are measured, the types of instruments available, their characteristics and some practical factors. For more detailed information on instruments, the reader is referred to Hanna (1985), USBR (1987), and to manufacturers of the instruments.

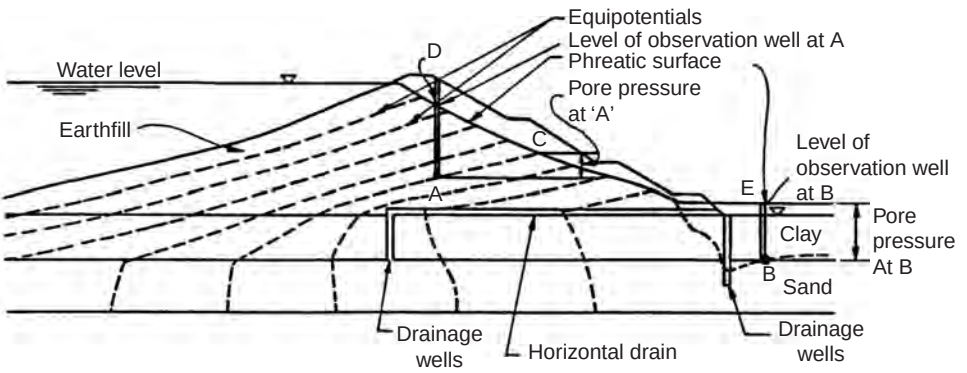


Figure 20.13 An example of embankment dam piezometric conditions.

20.4.4.1 *Why and where are pore pressures measured?*

There is a wide range of situations where it is necessary to measure pore pressures. Examples are given to highlight some of the principles involved in locating the piezometers to obtain meaningful answers.

Dam embankment

Figure 20.13 shows piezometric conditions in an earthfill dam, constructed on an alluvial foundation, consisting of clay over sand. The ratio of horizontal to vertical permeability (k_h/k_v) of the earthfill is 15:1. Pore pressures in the embankment and foundations are important from slope stability considerations, and for the potential for a 'blowout' or 'heave' condition to form at the toe of the embankment. Piezometers may be installed to check and monitor design assumptions.

The following points should be noted:

- An observation well, installed at A, with a sand surround and/or slots for its full length, will reach equilibrium at the phreatic surface at D.
- The pore pressure at A, i.e. the pressure that would be measured by a piezometer tip set at point A, is equal to the difference in elevation of A and the point where the equipotential through A meets the phreatic surface at C. Note that this may be much less than the difference in elevation from A to D, due to head losses in seepage flow through the earthfill.
- A slotted standpipe at B (with pipe extending above ground surface) will rise to the phreatic surface at E. In this case the pore pressure at B equals the head BE, because the pore pressure at B is higher than at any other point intersected by the standpipe. If the standpipe was cut off at the ground surface, water would flow from the pipe.
- It is the pore pressure in the embankment which is critical to stability, not the position of the phreatic surface. If k_h/k_v was only say 4, the equi-potentials would be steeper and the pore pressures for the embankment would be higher for the same

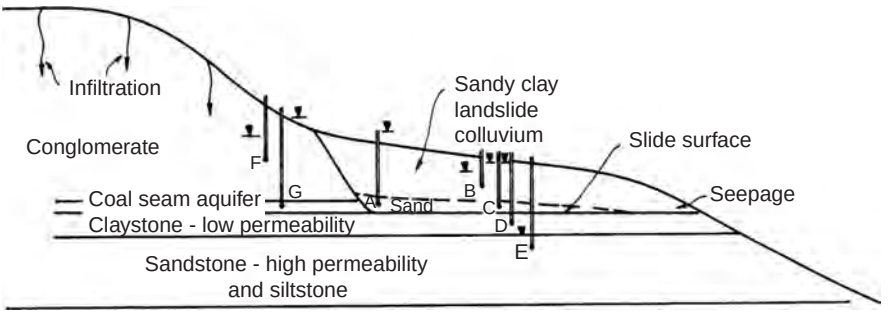


Figure 20.14 Landslide piezometric conditions.

phreatic surface. Hence pore pressures, not the location of the phreatic surface, should be measured.

Landslide

Figure 20.14 shows piezometric conditions in a landslide. Steady state piezometric levels are shown for piezometers installed at points A to G.

The following points should be noted:

- The factor of safety against sliding is very sensitive to the pore pressures. From a slope stability viewpoint it is the pore pressure on the slide plane which is critical, i.e. at A and C. Piezometers (B) in the landslide colluvium may give different pore pressures.
- Piezometers (D), installed in a borehole drilled into a low permeability layer below the slide colluvium, should be backfilled with sand past the slide plane, or the response time for the piezometer will be very slow and pressures different from that on the plane may be measured.
- If boreholes are extended into a more permeable layer as for piezometer E, low pore pressures may be measured. If the sand backfill is extended to the slide plane, water may drain towards the sandstone/siltstone and give lower pressures than actually are present at the slide plane.
- Piezometers installed in the low permeability conglomerate may take a long time to reach equilibrium and are likely to show lower piezometric levels than piezometers installed in the coal seam aquifer (see F and G).

Jointed and sheared rock

Figure 20.15 shows a jointed and sheared rock mass into which a cutting has been excavated. The low permeability shear zone and clay infilled joint affect the flow of groundwater towards the excavation.

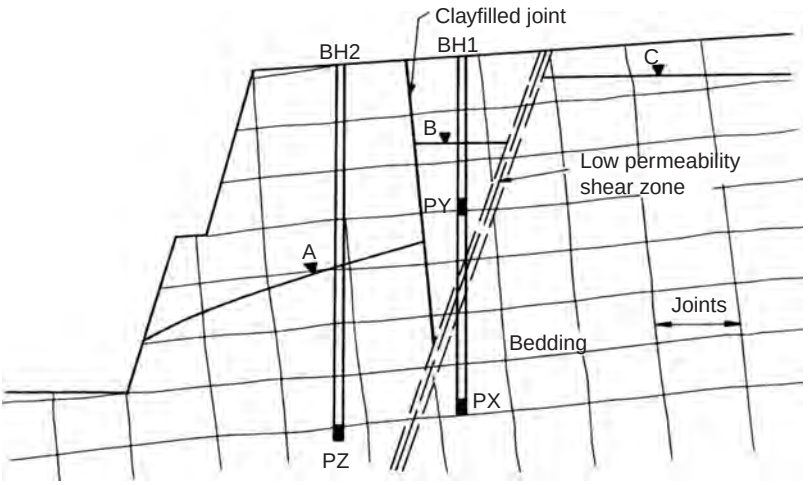


Figure 20.15 Piezometric conditions in jointed and sheared rock.

The stability of the cutting is determined by the strength of the joints, bedding planes and shear zone, which in turn is determined by the water pressure on these features.

If piezometers are installed in boreholes BH1 and BH2 as shown:

- Piezometer PX will read pressure due to phreatic surface C.
- Piezometer PY will read pressure due to phreatic surface B.
- Piezometer PZ will read pressure due to phreatic surface A.

In practice most rock masses are very complex, and it is not possible to simplify the model to the extent shown in Figure 20.15.

The geological factors which control groundwater movement must therefore be recognised when installing piezometers and several piezometers must be installed so that the critical condition can be defined. The piezometers should be installed with a permeable surround, which intercepts the joints and bedding planes along which groundwater flows.

Dewatering an excavation

Figure 20.16 shows an alluvial soil profile into which an excavation has been constructed. The groundwater has been lowered by well points and deep well pumps.

Monitoring of the effectiveness of the dewatering system, by installation of piezometers, would be needed to check overall slope stability and heave of the bottom of the excavation. To achieve this, piezometers would have to be installed in the silty sand and in the sand because the pore pressure regimes are different. Several piezometers would be needed in each soil unit, because of the rapidly varying pressures away from the well points.

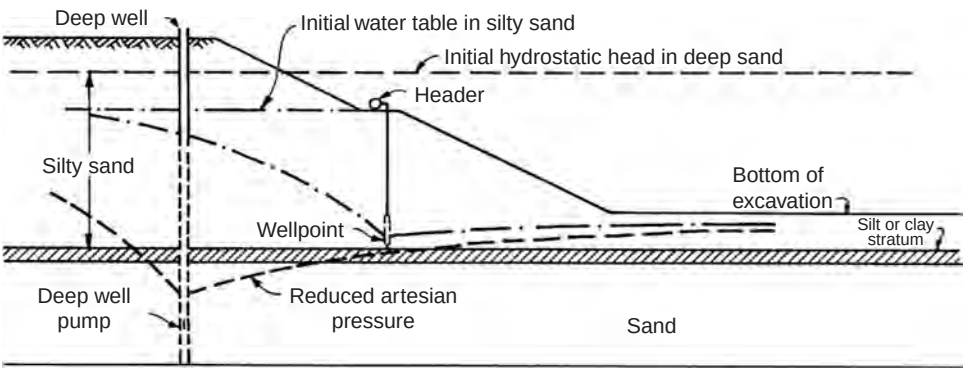


Figure 20.16 Example of dewatering an excavation.

20.4.5 Pore air and pore water pressure

The effective stress in a soil σ' is given by:

$$\sigma' = \sigma - u \tag{20.1}$$

where σ = total stress and u = pore pressure.

In partially saturated soils the pore pressure (u) is a function of the pore air pressure (u_a) and pore water pressure (u_w):

$$u = u_a - \chi(u_a - u_w) \tag{20.2}$$

where χ is a function of the degree of saturation. For fully saturated soils $\chi = 1.0$, and in a dry soil $\chi = 0$.

In partially saturated soil, pore air pressure is commonly greater than pore water pressure due to surface tension effects on the air-water meniscus. Hence, when measuring pore pressures in partially saturated soils, the piezometers must be constructed so that either pore air or pore water pressure is measured. This is achieved, in most cases, by using high-air-entry filters for the piezometer element which measures pore water pressure, and low-air-entry filters where pore air pressure is to be measured. These are constructed of porous stone, ceramic or bronze. The pores are very small and uniform in size and, when saturated with air free water, air cannot pass into the filter until the applied air pressure exceeds the pressure developed on air-water meniscus at the entrance of each pore in the filter. Hence, if fine 'high-air-entry' filters are used, the piezometer will measure pore water pressure and, if coarse filters are used, pore air pressure will be measured.

Air and other gases may also be dissolved in the pore water so even in saturated soils, air may enter the piezometer tip if it is too coarse.

In such cases, the air can lead to incorrect readings by lengthening the response time of the piezometer and forming air-water menisci in the tubes and standpipes which affect the measured pressure.

Sherard (1981) includes a discussion of the differences between pore air and pore water pressure in dam construction. He concludes that:

- In granular cohesionless soils $u_a \approx u_w$
- In clay soils $u_a > u_w$, but not by more than 2–3 m of water head where $u_w > 5$ m water head. At higher pressures the difference rapidly decreases.
- The difference is always highest during construction when the soil is partially saturated.

Sherard quotes case histories where low air entry and high air entry tips have been used adjacent each other, with only small differences in readings. However, this experience is in large dams where pore pressures are large in magnitude. In smaller embankments the differences may be significant.

Sherard (1981) recommends use of high air entry tips for both hydraulic and vibrating wire piezometers. He suggests use of tips with an entry pressure of 500-600 kPa where these are available. He recommends use of tips with lower air entry pressures for pneumatic piezometers, the entry pressure being selected to be just greater than the anticipated soil suction (in partially saturated soils). This prevents transfer of water to the surrounding soil from the piezometer tip, while still allowing the transfer of the small volume of water that is required to operate the piezometer.

20.4.5.1 *Fluctuations of pore pressure with time and the lag in response of instruments*

In most practical situations the pore pressure being measured varies with time, e.g.:

- Pore pressures in a dam vary with reservoir level.
- Pore pressures in a landslide will vary with rainfall.
- Pore pressures around a pump well will vary with rate of pumping and the time lag between spells of pumping.
- Pore pressures under an embankment on soft clay will vary with the height of the embankment and degree of dissipation by consolidation of pore pressures induced by the embankment.
- Dynamic loading, e.g. earthquakes, may induce an increase in pore pressure.

In all cases, the piezometer which is selected should be constructed so that the time taken for the piezometer to respond to the change in pore pressure is sufficiently short to give a meaningful measure of the actual pore pressure.

In a Casagrande piezometer (Figure 20.18) the piezometer responds to a change of pore pressure by flow of water into or out of the piezometer. This is a function of the geometry of the piezometer, the permeability of the soil and the change in pore pressure.

The response is discussed in Hanna (1973, 1985) and Brand and Premchitt (1980a, b).

Table 20.11 is reproduced from Standards Association Australia (1979) and gives times for 95% and 99% equalization of Casagrande type piezometers with a 73 mm 'piezometer tip' and 9.5 mm inside diameter standpipe.

Table 20.11 95% and 99% equalization time lag for Casagrande Piezometers (Standards Association Australia 1979).

Material	*Sand*	*Silt*			*Clay*		
Permeability cm/s	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Average time lag for 95% equalization	12 s	2 min	20 min	3.5 h	36 h	14 days	150 days
Average time lag for 99% equalization	18 s	3 min	30 min	5.2 h	54 h	21 days	225 days

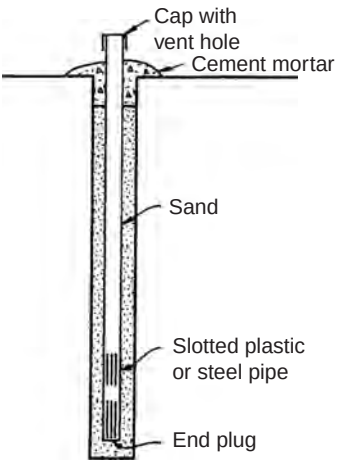


Figure 20.17 Observation well.

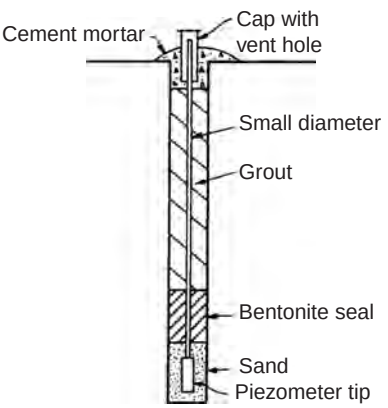


Figure 20.18 Casagrande type piezometer.

Because of the small volume of water flow required to activate hydraulic, pneumatic, vibrating wire and strain gauge piezometers, their time lags are very small compared to that of Casagrande piezometers.

For hydraulic piezometers (Figure 20.19) the 'response' time depends on whether the piezometers have to be flushed, in which case time has to be allowed for pressures to re-establish equilibrium due to the flushing affecting pore pressures around the piezometer tip. It also depends whether individual pressure gauges are provided for each piezometer, or whether a master gauge is used. In the latter case, time must be allowed for equilibrium to be achieved.

Pneumatic piezometers (Figure 20.20) have a small but finite response time, as a small displacement of the diaphragm is needed. The response time varies depending on whether high or low air entry tips are used.

Vibrating wire (Figure 20.21) and strain gauge piezometers have virtually zero response time, as there is no transfer of water involved.

20.4.5.2 *Types of instruments and their characteristics*

Observation well

As shown in Figure 20.17 an observation well, or 'open standpipe', consists of a slotted plastic, reinforced fiberglass or steel pipe attached to a standpipe, installed in a borehole and surrounded with sand. Porous tips, as described below for Casagrande piezometers, may be used. When slotted pipe is used, it is often wrapped in filter fabric to prevent clogging by the sand. The hole is sealed at the top with mortar to prevent surface water flowing in. The water level in the well is measured with a water level dip meter.

An observation well normally measures the maximum water pressure intersected in the hole. It does not measure pressure at a point, and does not account for flownet effects (see Figure 20.13). In some situations, with confined aquifers of varying permeability, an observation well may not measure the maximum pressure, with water from the high pressure zone flowing into a more permeable lower pressure zone.

The time lag for observation wells may also be large, since a relatively large flow of water is required for the well to reach equilibrium pressure change.

In most situations observation wells will not be adequate, and piezometers should be installed.

Observation wells have proved to be valuable when checking how the groundwater surface has behaved at a dam site through all phases of the project development where the actual piezometric pressures are not so critical.

Casagrande piezometer

Figure 20.18 shows a Casagrande type piezometer. These consist of a porous tip which is embedded in a sand filter, and sealed into a borehole with a short length (usually 1 metre) of bentonite. The hole should be backfilled with cement bentonite grout. In dam cores grouting should be staged to avoid inducing hydraulic fracture. The standpipe is kept as small a diameter as practical to keep the response time to a minimum. Dunnicliff (1982) recommends the standpipe be not smaller than 10 mm diameter so that gas bubbles will not be trapped. In any case, this is a practical minimum for use

of a dip meter to measure the water level. The sand filter acts as a large collector of water compared to the small diameter standpipe.

The bentonite seal may be formed using dry pellets which are available commercially, or by making 25 mm diameter moist balls of powder bentonite. These are dropped down the hole, possibly tamped to compact and allow checking of the position and depth of the seal.

Casagrande, and all standpipe piezometers, measure pore water pressure (u_w). They are potentially accurate, depending on how carefully the water level is determined. The time lag can be determined as outlined in Section 20.4.5.1. Time lag may be a problem in lower permeability soils, and where the pore pressure is changing rapidly.

The Casagrande type piezometer is low cost, simple to construct with readily available equipment and materials. They are self de-airing, measure pore water pressure and have a long, satisfactory performance record. Disadvantages include susceptibility to damage by construction equipment or animals and by consolidation of soil around the standpipe, possible leakage from the surface runoff if the grout seal is not adequate, susceptibility to freezing and inability to measure pressures higher than the level of the standpipe or pressures lower than atmospheric. If porous filter tips are used, these may clog with chemical deposits with flow in and out of the tip. Reading is simple but relatively time consuming. Auto-data logging is not practicable.

Hydraulic twin tube piezometer

Figure 20.19(a) shows an Imperial College (Bishop) type hydraulic tube piezometer. Variations of the principle include the USBR type shown in Figure 20.19(b).

These consist of a porous tip, which is usually installed during construction (e.g. in a dam embankment), from which two tubes are led to a convenient terminal measuring point where gauges are used to record the pressure. The total pressure equals tip pressure, plus elevation difference from the tip to the measuring gauge.

Two tubes are provided so that air (gas), which is trapped in the tubes or which enters through the porous tip, can be flushed from the system using de-aired water. The tips are constructed of fine porous ceramic, stone, aluminium or sintered bronze. The tips are required to be strong enough to withstand the total pressure, and fine enough to prevent clogging by the soil. More importantly, the tips must be 'high-air-entry' to limit entry of air into the system. Commonly, high-air-entry tips can withstand an air pressure of 100 kPa to 200 kPa, but may be up to 500 kPa to 600 kPa.

Coarse, low-air-entry tips are not used with hydraulic piezometers because they allow air to enter the tip, the piezometer and tubes. This leads to pore air pressure rather than pore water pressure being measured in partially saturated soils, a longer response time for the piezometer, and incorrect elevation pressure adjustment if the air enters the tubes. The high-air-entry tips are only effective if they are completely de-aired before installation. This is usually done by boiling for 2 hours to 6 hours. Sherard (1981) suggests 12 hours to 24 hours boiling.

The tubes leading to the measuring gauge should be constructed of Nylon 11, covered by polyethylene. The Nylon 11 is impervious to air, the polyethylene impervious to water, and the combination is impervious, strong, flexible and not affected by chemical attack.

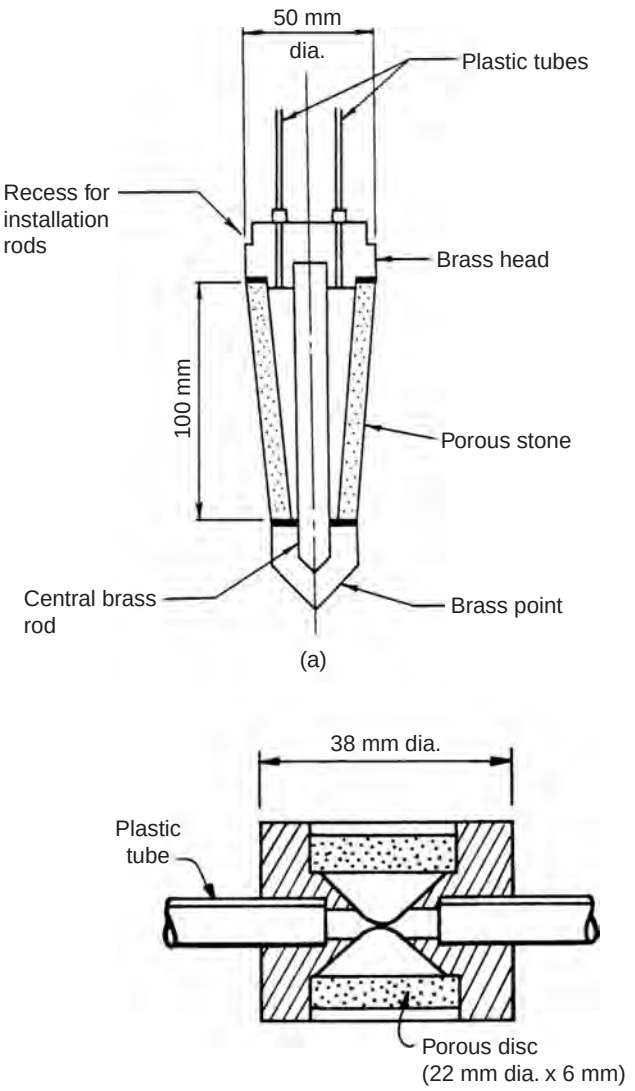


Figure 20.19 (a) Bishop and (b) USBR type hydraulic piezometers (ANCOLD, 1983).

Despite the use of high-air-entry tips, and the proper tubing, gas may still accumulate in the tubes, and periodic flushing with de-aired water is necessary. This should be done under back pressure to ensure that negative pore pressures are not induced. Some organisations add bacteriacides and algacides to the water, to prevent algae growth blocking the tubes.

Some organisations use flow meters on the inlet and outlet lines as an aid to flushing. A flow of 300 ml/min is maintained to prevent air becoming trapped in high

spots in the lines. It is also good practice to flush before all readings, since equal pressures on the lines can occur with air in both lines.

Eagles (1987) reports that if the entry to the piezometer is smaller than the tubing (e.g. 1.5 mm to 2 mm compared to 5 mm), crystal growth from chemicals dissolved from the tubing can cause blockage and it is important that the entry be the same diameter as the tubing.

The tip pressure is measured using Bourdon gauges, measuring manometers, or pressure transducers in the terminal structure. Most installations use Bourdon pressure gauges – either one gauge for each piezometer, with a master gauge to check, or a master gauge with a switching system. Typical layouts are given in ANCOLD (1983) and USBR (1987). When using a master gauge, it is necessary to estimate the tip pressure and pressurize the gauge to this pressure before connecting the piezometer. Any imbalance will involve flow of water into or out of the piezometer tip, with resultant lag time, and possibly influence the pressure, particularly if the soil is partially saturated and water flows out of the piezometer tip. Use of a gauge for each piezometer with a check master gauge overcomes this.

The terminal structure and tubes must be located so that no more than about 5 m negative head is generated, or cavitation will occur in the tubes, giving incorrect readings.

In embankment construction, the tubes have to be laid loosely in trenches to avoid damage by construction equipment and differential settlement effects. Trenches are usually filled with sand, with bentonite clay or clay cutoffs to prevent water seepage along the trenches. (See examples in and USBR (1987). However in many dams the trenches were backfilled with loose soil similar to the dam core, rather than sand, and piezometers were located close to the upstream face of the earth core. This leaves a potential weakness in the dam from an internal erosion and piping point of view, since high gradients may occur from the upstream face of the core to the first piezometer and, if erosion were to initiate, it would quickly extend through the loose soil. This would potentially put high gradients across the bentonite/soil cutoffs.

The authors are not aware of any cases where this has happened, but strongly recommend against this practice for new dams.

Provided that high-air-entry tips are used, and the tips installed with intimate contact with the soil, hydraulic piezometers will read pore water pressure. Sub-atmospheric pressures up to about 50 kPa to 70 kPa can also also be read.

Twin tube hydraulic piezometers are relatively simple and medium cost to construct, although when the cost of the terminal house (which may have to be dewatered to position it satisfactorily, to avoid excessive negative pressures) is included, the overall cost may be comparable with vibrating wire and pneumatic systems.

Auto-data logging is possible by using electrical pressure transducers. Hydraulic piezometers have a medium long term reliability (15 years to 20 years in many cases), and have been the favoured instruments in British and Australian dams. However most hydraulic piezometers more than 30 or 40 years old are now not functioning. They were used in USBR dams until 1978 when pneumatic piezometers were introduced, because they are easier to read and maintain in good working condition. Disadvantages include 'growths' in the tubes; damage of tubes by differential settlement; the need for an elaborate terminal house, often requiring dewatering; skill and care to flush the piezometers and to keep them operating and the need to avoid high

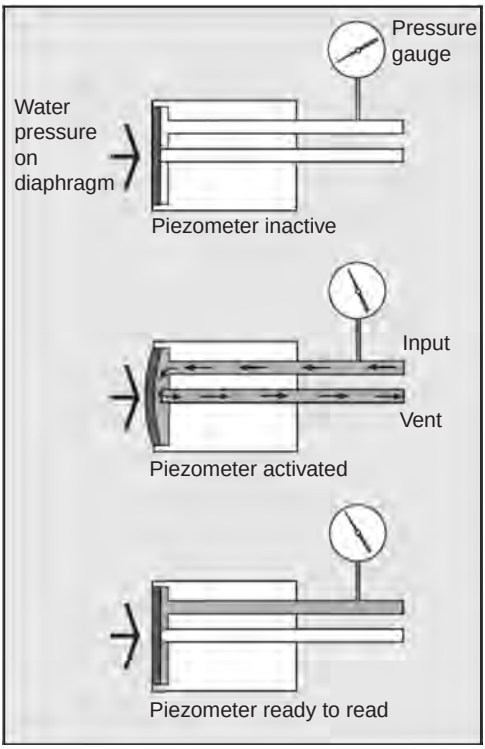


Figure 20.20 Operating principle of a pneumatic piezometer (Slope Indicator Co.)

sub-atmospheric pressures in the lines. Particular advantages are the ability to flush air from the piezometer tip if necessary, hence ensuring measurement of u_w, ability to measure moderate sub-atmospheric pressures, and the possible use of the piezometer to carry out *in situ* permeability and hydraulic fracture tests of the soil. As pointed out by Sherard (1981), hydraulic (and pneumatic and vibrating wire) piezometers should not be installed in a surround of sand. Doing so only slows the response time if the sand surround becomes partially saturated and since the piezometers are essentially no flow, a large sand surround has no benefit in collecting water.

The authors know of no organization installing hydraulic piezometers in new dams and do not recommend them.

Pneumatic piezometer

Figures 20.20 and 20.21 demonstrates a pneumatic piezometer consists of a piezometer tip, from which two tubes are led to a convenient terminal measuring point. Pressure is measured by applying pressurised gas (usually dry nitrogen) on the inlet tube. The diaphragm is moved, and gas escapes to the outlet tube where it can be observed. The pressure on the inlet tube is then reduced until the diaphragm closes (no flow from the outlet tube). The pressure at which the diaphragm closes is the pore pressure (after

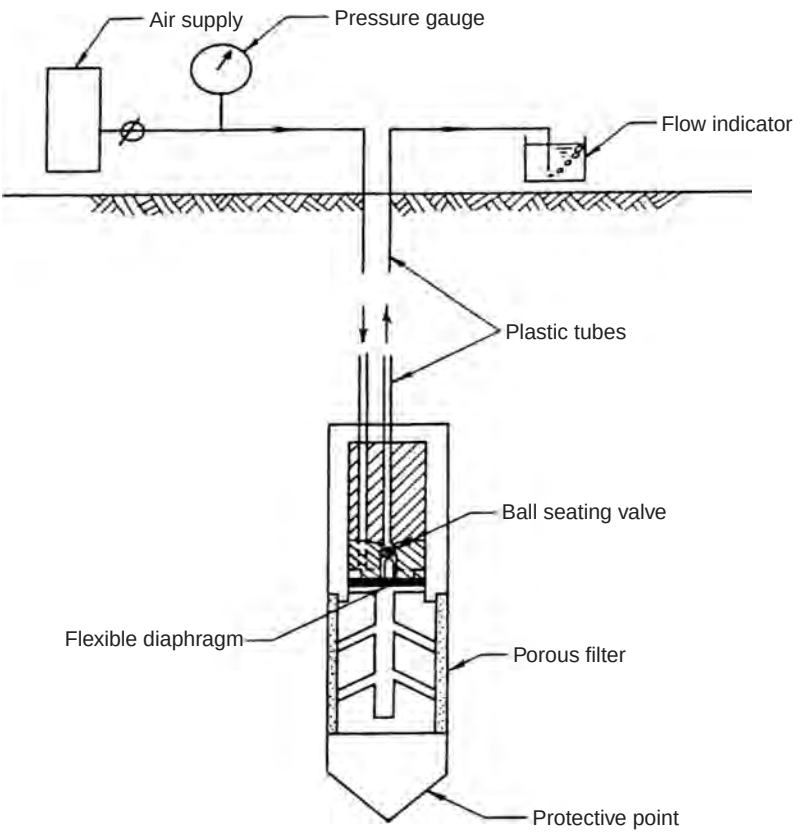


Figure 20.21 Pneumatic piezometer (from ANCOLD, 1983).

correction for any closure spring effect). Some manufacturers measure the pressure on opening of the diaphragm but most use the principles outlined above.

The piezometers are fitted with high- or low-air-entry tips. Sherard (1981) discusses at some length the relative merits of using high- or low-air-entry tips. He concludes that if low-air-entry tips are used in unsaturated soil, the water in the tip will be sucked out by capillary action and the piezometer will measure pore air pressure u_a. However, if high-air-entry tips are used, the response time during reading may be affected, and the piezometer may read an erroneously high pressure. This is because of the small volume of water displaced by the action of the diaphragm. Sherard suggests using tips with an air entry value just above the soil suction pressure. This is not a problem when the piezometers are being used in saturated soils and, since it is easier to de-air low-air-entry tips, these are often used. Where high pore pressures are to be measured the pore air and pore water pressures are similar so the choice of tip is not critical.

The tubes should be of the same construction as for hydraulic piezometers, i.e. Nylon 11 sheathed in polyethylene, as this gives strong, stiff tubes, impervious to ingress of water and leakage of air. If water does enter the tubes, flushing with dry

nitrogen is necessary. The use of lighter tubing can result in problems with crushing and kinking and the savings in cost may not be warranted.

The pressure is usually read by a portable readout which is connected to the tubes with quick-connect couplings. Either a Bourdon or digital type gauge is used. The latter can be adapted to auto-data logging with the terminal permanently installed and programmed to interrogate each gauge in turn at predetermined intervals.

The terminal pressure gauge can be located at any elevation relative to the piezometer and tubes can be laid in any configuration. In embankments, the tubes must be laid in trenches to avoid damage by construction equipment and looped, coiled etc. to allow for differential movement. As for hydraulic piezometers, these trenches are a potential point of weakness. The piezometers are installed into existing dams using a similar system to that shown in Figure 20.18, with piezometer in a sand surround, with a bentonite seal and cement/bentonite grout or in contact with the soil if negative pre pressures are to be measured.

Because of their method of construction and reading, pneumatic piezometers cannot be used to measure sub-atmospheric pressures.

Pneumatic piezometers are relatively simple and low cost construction, and the terminal gauge house requirement is much simpler than for hydraulic piezometers. Pneumatic piezometers have performed satisfactorily over 10 years to 15 years in the USA (Sherard, 1981) and were then the preferred instrument for dams in that country because they are easier to read, easier to install and auto-data logging is simple. Less skilled operation is needed than for hydraulic piezometers. There was some reticence to adopt them for dams in some other countries, including Australia. Where used, their performance has been acceptable, but many cease to work after some years. They have been used in boreholes in rock (and in concrete dams) by installing with a packer, allowing replacement if this was necessary. The time for taking measurements is dependent on the readout equipment and the length of the tubes to the piezometer. Sherard (1981) quotes read times of 1 minute to 2 minutes for short tubes, but up to 10 minutes to 20 minutes for very long tubes. He indicates this was reduced to 3 minutes to 5 minutes by initial rapid gas filling. There are no problems of freezing with pneumatic piezometers.

Disadvantages include long measurement time for long tubes and high and sub-atmospheric pressures cannot be read.

Vibrating wire piezometer

Figure 20.22 shows the principle of a vibrating wire piezometer (also known as an acoustic piezometer).

These consist of a piezometer tip from which an electrical cable leads to a convenient terminal measuring point. The piezometer consists of a porous tip, as in a pneumatic piezometer with a stiff metallic diaphragm. The diaphragm is attached to a prestressed wire. When the diaphragm deflects with changes in pore pressure, the tension in the wire changes. The natural frequency of the wire is dependent on the tension and is measured by plucking the wire, using an electromagnet. The wire then vibrates in the magnetic field of a permanent magnet, causing an alternating voltage of the same frequency as the wire, and this can be measured at the measuring point.

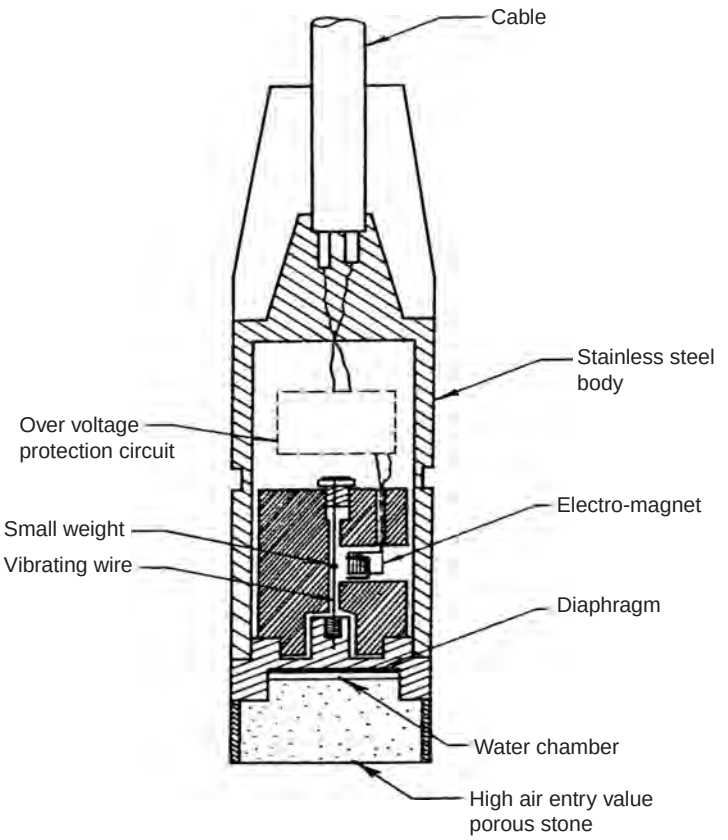


Figure 20.22 Vibrating wire piezometer – Maihak type (from ANCOLD, 1983).

Hence, by measuring the natural frequency of vibration of the wire and calibrating this against pressure on the diaphragm, the instrument can be used to measure pore pressure.

The instruments are essentially no flow devices and, therefore, have a very short response times. Sherard (1981) suggests that they are best fitted with high air entry tips to limit entry of air into the piezometer and ensure that pore water pressure is measured. A digital readout is usually provided and the instruments are readily suited to auto-data logging. The readout unit can be located at any elevation relative to the piezometer and up to several kilometres away. The wires can be laid in any configuration, but provision should be made for differential movement. In existing dams, vibrating wire piezometers are installed in boreholes and sealed in place as shown in Figure 20.18. Provided they are installed with intimate contact with the soil, vibrating wire piezometers can be used to measure sub-atmospheric pressure.

Vibrating wire piezometers are more expensive than hydraulic or pneumatic piezometers, but have been used widely in European, USA, Canadian and Australian

dams, with reliable performance for up to 20 years (Sherard, 1981). Australian experience is that most piezometers more than 30 years old are no longer functioning. They are the easiest and quickest to read of all piezometers and are not susceptible to freezing.

There are varying opinions on whether they are susceptible to long term drift, due to creep of the diaphragm or tension wire. Sherard (1981) presents information suggesting this is not a problem. Australian experience is that it can be. Other advantages are that, by using high-air-entry tips, the saturation of the tips can be checked by observing the development of negative pressure as the tip is allowed to dry slightly and, in partially saturated soils, water will not be sucked from the tip into the soil (unlike hydraulic piezometers which may wet the soil during the flushing operation).

Disadvantages have included damage by lightning strike, particularly but not only during construction when the cables are exposed. This has been overcome by shielding cables, earthing, and provision of overvoltage protection. Stray currents from nearby power stations have also caused problems.

Strain gauge piezometer

Strain gauge piezometers are similar in construction to the vibrating wire piezometer, but the deflection of the diaphragm is measured by either bonded electrical strain gauges or Carlson type unbonded strain meters. Pore pressures are recorded as changes in resistance. The instruments are not as widely used as vibrating wire piezometers, mainly because of the possibility of drift in the long term and with temperature. However, the instruments have the advantage of being easy to read, easily auto-data logged and independent of the relative location of the piezometer tip and readout unit. As with vibrating wire instruments, they are susceptible to damage by lightning strike and overvoltage protection is needed.

20.4.6 Should piezometers be installed in the cores of earth and earth and rockfill dams?

Many large dams built from the 1960s to 1990s have piezometers installed in the dam core. These were usually installed as the dam was constructed and the piezometer tubes (or wires for vibrating wire piezometers) were taken to the downstream side in trenches as described above. In some dams, the wires or pneumatic piezometer tubes were taken to the top of the dam in groups in vertical riser pipes.

Either method leaves a potential weakness in the dam and some senior dam engineers, including the authors, are reluctant to put such potential weaknesses into a dam core, because of the chance that internal erosion and piping may initiate along the trenches/risers. It is arguable whether, for a well-designed and constructed dam with filters downstream of the core, it matters what the pore pressures are.

Having said that, the authors have on a number of occasions found the long term trends in pore pressures valuable in assessing slope instability and potential piping problems.

If piezometers must be installed, it is suggested the tubes/wires are laid in a winding fashion in a trench backfilled as recommended in USBR (1987) with well compacted dry mixture of bentonite and filter sand (1 part bentonite, 3 parts filter sand by dry

weight). The degree of compaction should match that of the adjacent fill. Seepage cutoff trenches should be provided, extending at least 0.6 m into the adjacent fill, and 0.3 m wide, where zone contacts are crossed between each piezometer in the earthfill core and at not more than 15 metre intervals.

The bentonite sand backfill should be compacted because otherwise a gap may form at the top of the trench as the sand densifies under compaction equipment and on saturation. This may make it more likely for tubes/wires to be broken but the potential for a gap must be avoided.

The sides of the trench are likely to desiccate and crack, and cracked material should be removed before backfilling.

Piezometers should not be installed too close to the upstream face of the core – say no closer than a quarter of the core width and preferably not in the upstream half for narrow cores.

Where existing piezometers have ceased to function, it is usually better to go without the information, than to run the risk of damaging the core during drilling and backfill grouting of the holes needed to install new piezometers. If there are signs of deterioration necessitating installation of piezometers in the core, these should be installed in boreholes using dry drilling techniques, such as hollow flight augers or sonic drilling so as to avoid hydraulic fracture of the core. The backfill cement/bentonite grout should be installed in short lengths, using a tremie system, with time between stages in the backfill to allow the grout to develop strength, so hydraulic fracture will not be caused by the grout. This work should be supervised by experienced geotechnical professionals familiar with dam engineering.

20.4.7 Displacements and deformation

Internal displacements and deformations were often monitored on larger earth, earth and rockfill and concrete face rockfill dams. This allowed confirmation of design assumptions, particularly relating to long-term settlement of earthfill and foundations, and to concrete face designs.

Hanna (1985), ANCOLD (1983) and USBR (1987) give details of the instruments used. Up to date information can be obtained from instrument manufacturers. A brief overview of the types of instruments used follows.

20.4.7.1 *Vertical displacements and deformation*

Vertical displacements and deformations are measured using mechanical or magnetic devices which are either embedded in the fill as the dam is constructed, or installed in boreholes. Figure 20.23 shows the principle of the USBR type mechanical device.

The base tube is installed in the dam foundation and a cross-arm anchor placed in the dam foundations. Telescoping tubing is placed in the fill as the dam is constructed, with cross-arm anchors being installed at intervals to allow differential settlement within the fill to be measured. Typically cross-arms will be spaced at 2 m or 3 m intervals. Settlement readings are obtained by survey of the top of the coring and lowering a measuring torpedo down the tube. The torpedo is provided with spring-loaded arms which engage the lower ends of the cross-arm pipes. It is supported on a measuring tape, and can allow accuracy of measurement of ± 3 mm (accuracies as small as ± 1 mm

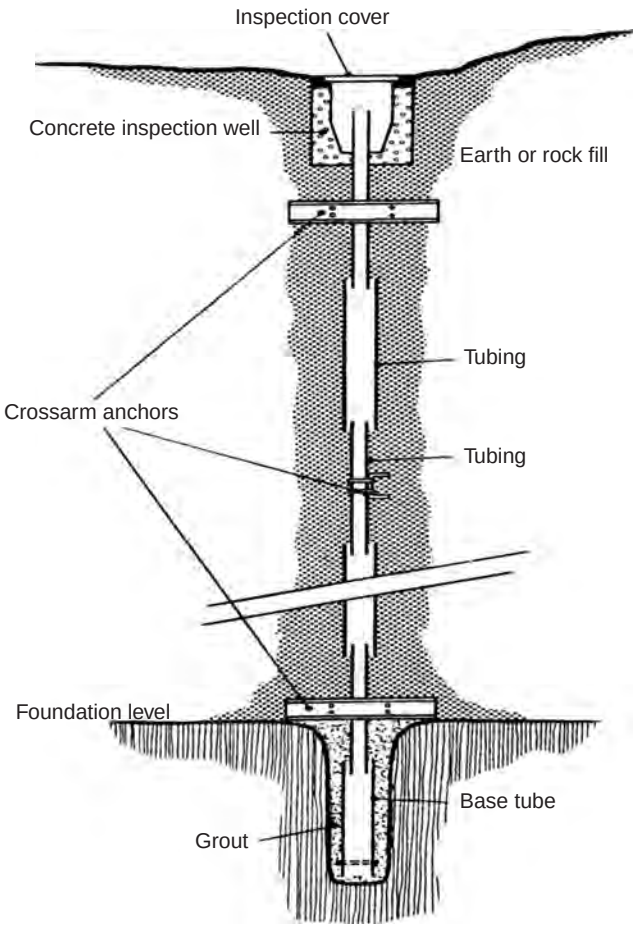


Figure 20.23 USBR type cross-arm settlement gauge.

are claimed). Settlements as much as 15% can be accommodated and the testing can be installed to depths of at least 100 m. The tubing does not have to be vertical (Soil Instruments Ltd., 1985, indicate ±25° from vertical is allowable).

The USBR instrument cannot be installed down a borehole, only in fill as it is constructed.

Figure 20.24 shows a magnetic version of the instrument which uses ring magnets instead of the cross-arms.

In fill, ring magnets are placed in the ground at predetermined intervals, in the same way as the cross-arms are for the USBR device. In boreholes a spring-loaded magnet is lowered through the grout supporting the borehole, and is held to the walls of the hole by the spring and the grout. The grout has compressibility equal to or greater than, the soil or rock surrounding it. The probe is lowered down the tube and senses the position of the magnets. If suspended by a tape, the accuracy of measurement is

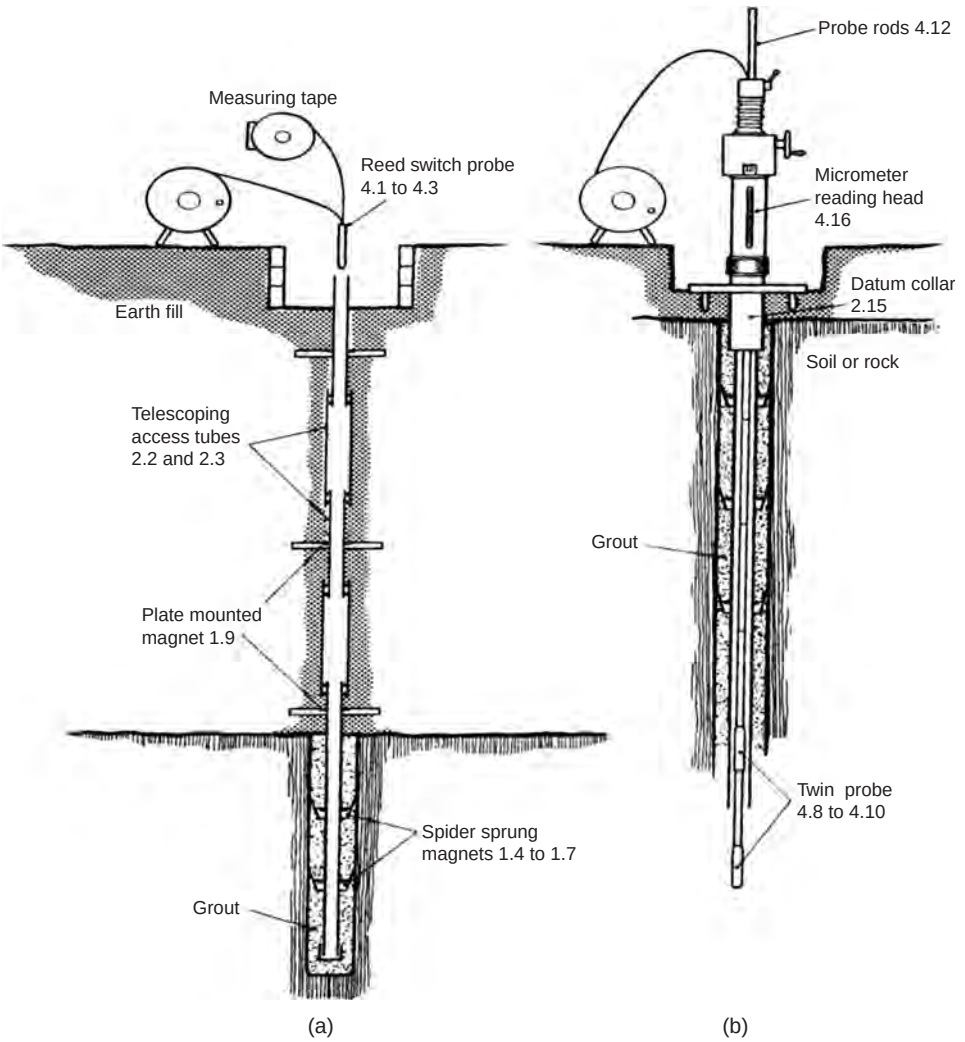


Figure 20.24 Magnet (Soil Instruments Ltd, 1985) settlement gauges: (a) Magnets embedded as earthfill is placed; (b) Magnets placed in a borehole.

±1 mm. Soil Instruments Ltd. (1985) indicates that a rod-mounted version fitted with a micrometer device to help position the magnet can read to ±0.1 mm.

The tape suspended device can be used in the same conditions as the USBR device, i.e. in near vertical holes. The rod-mounted device can be used in any orientation, so can be used to measure horizontal displacements, as well as vertical.

While these instruments have provided useful information on dam deformations, it is questionable whether they should be installed in earthfill cores, because it is very difficult to compact around the riser, and these can be a source of local collapse settlement

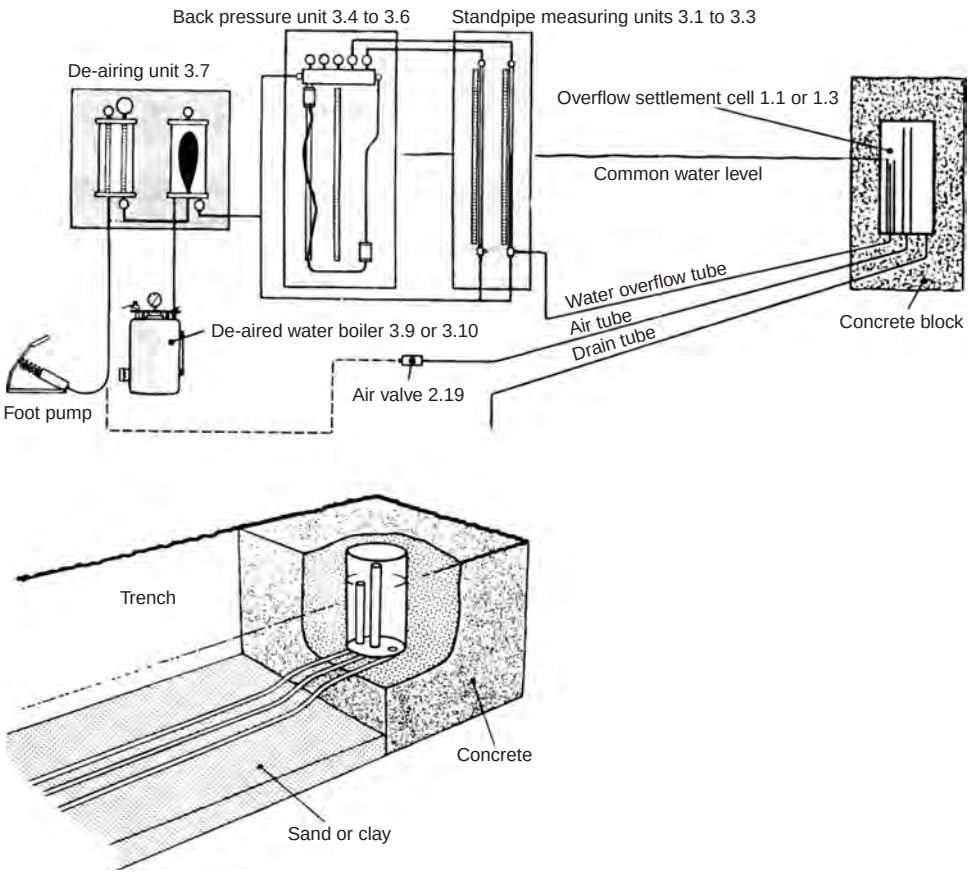


Figure 20.25 Hydraulic overflow settlement equipment (Soil Instruments Ltd, 1985).

on saturation giving a potential defect in the dam from an internal erosion and piping perspective.

Figure 20.25 shows the principle of a hydraulic overflow settlement cell.

This works on a simple U tube principle. The cell is cast into the fill and connected by a water tube to a graduated standpipe. A second drain tube allows surplus water to flow from the cell and a third 'air' tube maintains the interior of the cell at atmospheric pressure. Compressed air is used to clear the air tube and drain tube, then the water tube is filled with de-aired water, pumping sufficient water to remove any bubbles in the water tube. After allowing time to reach equilibrium, the level of water in the standpipe is the same as that at the cell. The range of the instrument can be extended by applying small back-pressures to the standpipe.

The method is very simple and inexpensive and, according to Soil Instruments Ltd., gives an accuracy of the order of ±1 mm to ±5 mm, depending on the back pressure being applied. Hanna (1985) indicated an accuracy of ±2 mm. Tube lengths up to 300 m can be used. ANCOLD (1983) gives details of such a device, which uses

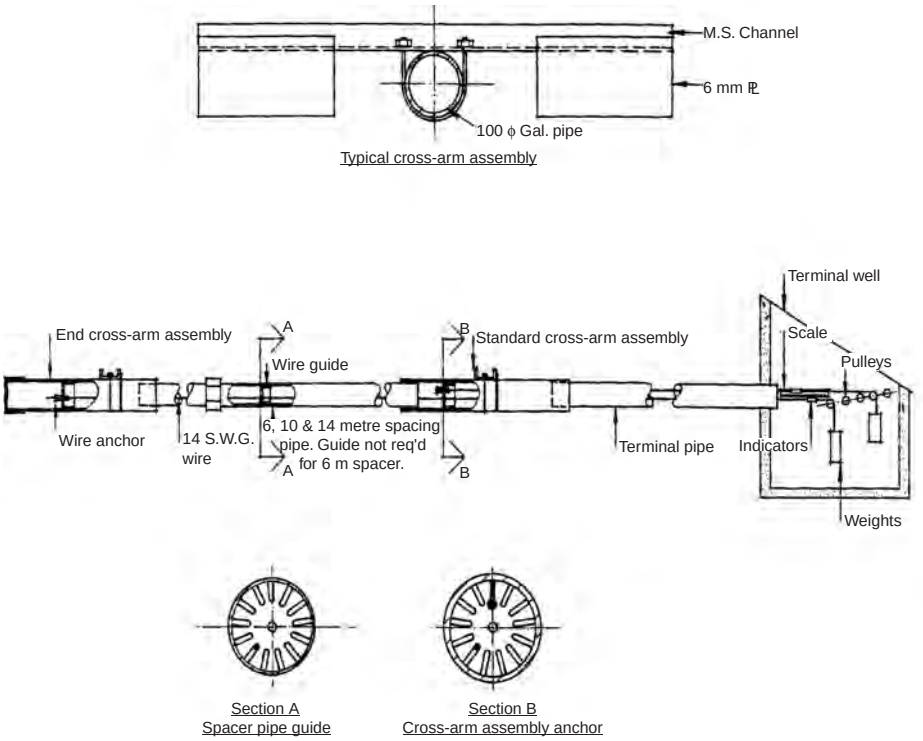


Figure 20.26 Long wire extensometer (ANCOLD, 1983).

two cells at the same location. These are at slightly different levels, allowing checking of whether there is air in the lines, and rotational movement of the cell.

These devices are used in rockfill zones where it would be difficult to install cross arm devices, but may be used in earthfill. Hanna (1985) and Soil Instruments Ltd (1985) describe the use of pneumatic and vibrating wire versions of the hydraulic settlement cell.

Again, in earthfill cores there may be a potential defect due to poor compaction and it is questionable whether they should be installed.

20.4.7.2 *Horizontal displacements and deformations*

Extensometers

On some very large dams, horizontal displacements and deformations are measured using mechanical and electronic devices embedded in the dam fill as the dam is constructed.

Their use is usually combined with hydraulic settlement gauges, so that the settled profile can be determined. The mechanical instruments are installed in rockfill zones, it being considered unwise to place them through the earth core as they may form a preferred leakage path.

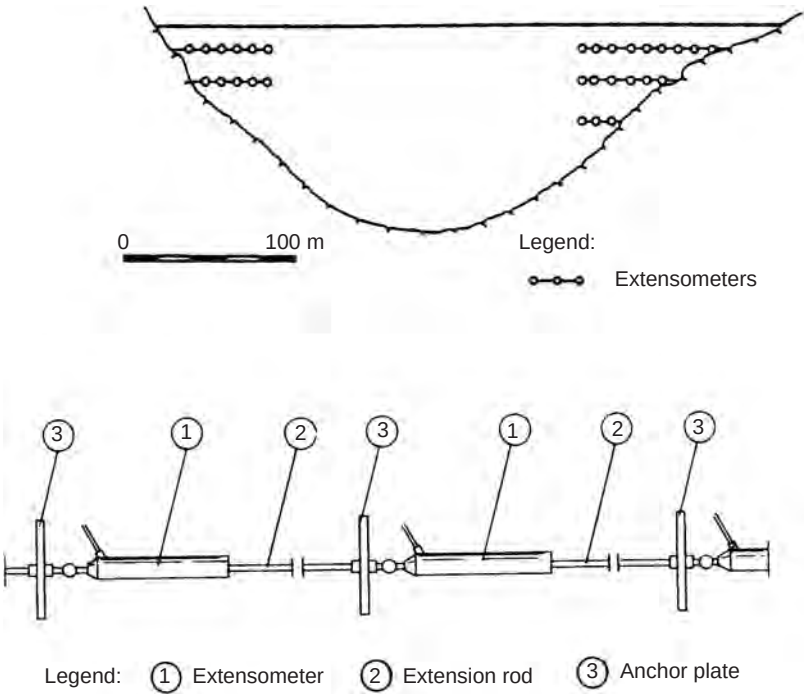


Figure 20.27 Use of extensometer to measure strain parallel to the dam axis (ICOLD, 1989).

Figure 20.26 shows a mechanical device which uses long wires, anchored to cross arms embedded in the dam. The wires are held at constant tension, and displacements measured at the terminal well, which is situated on the face of the dam.

The wires are housed in telescoping segments of pipe. Corrections for overall translation is made by surveying the position of the terminal well. ANCOLD (1983) indicate the accuracy to be ± 1.5 mm.

Soil Instruments Ltd. (1985), and Hanna (1985), describe other extensometers which can be used to measure horizontal displacements and strains. These include devices which use vibrating wire and resistance potentiometers to do the measurements.

Figure 20.27 shows the use of such devices to measure deformations in the earth core of a dam parallel to the dam axis. In this case, several extensometers are installed with extension rods connecting the anchor plate and the vibrating wire extensometers. This data can be valuable when assessing the performance of the dam. The authors have experience of quite large strains being measured which have supported the conclusion that the filters, which did not meet modern standards, should be upgraded.

Inclinometers

Lateral movements in dams can also be measured by use of borehole inclinometers. The inclinometer casing may be installed during construction of the dam, or after, in the event that excessive displacements are observed. If the casing is installed on a

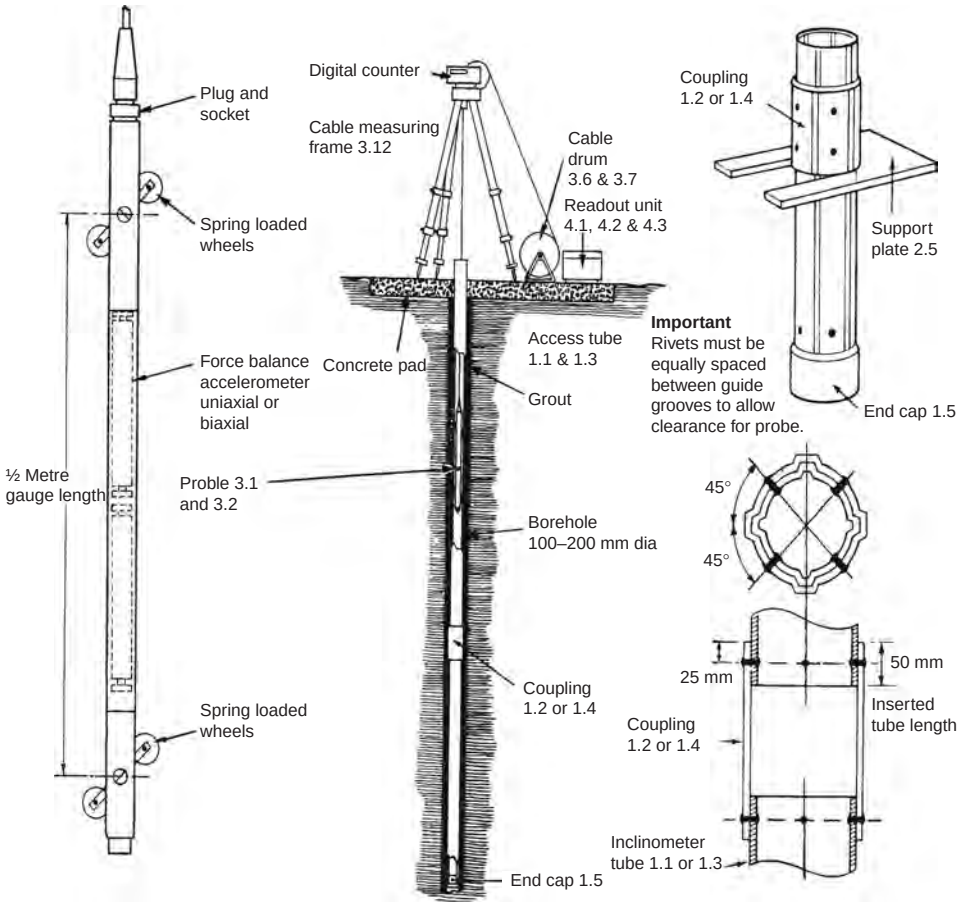


Figure 20.28 Borehole inclinometer (Soil Instruments Ltd., 1985).

slope, e.g. along the face slab of a concrete face rockfill dam, horizontal and vertical displacements can be determined.

Most inclinometers are similar to that shown in Figure 20.28, in that the inclinometer is passed down casing which is grooved to maintain the orientation of the sensor probe.

The tilt of the sensor is measured and recorded by servo-accelerometers located within the probe. Where two accelerometers are used they can sense the inclination of the tube in two directions at right angles to each other. Processing of the signal at the surface allows display of the angular rotation, or displacement normal to the casing in the two directions. Settlement gauge magnets may be installed around the casing to allow assessment of axial deformations.

The casing may be plastic or aluminium and should be able to telescope, to allow for differential settlement. Soil Instruments Ltd. (1985) indicate a resolution

of ±0.1 mm. Hanna (1985) indicates that careful calibration and maintenance is necessary to achieve a high precision.

The authors' experience is that a high degree of reproduceability is possible, although considerable care must be taken with instruments significantly off the vertical. The authors also feel that inclinators are best utilised to show distinctive trends in shear displacement rather than small displacements of 1–2 mm.

Hanna (1985) indicates that the grooves in the casing may spiral, and individual lengths need to be checked for this.

ANCOLD (1983) describe an inclinometer which can be used in smooth pipe. It contains two servo-accelerometers. The device was developed for monitoring displacements of the face of concrete face rockfill dams and has an accuracy of ±3 mm.

Borehole inclinometers are a valuable tool for investigating dams with potential instability, or large internal deformations. However, as for piezometers, great care must be taken in drilling the holes using dry drilling methods and grouting around the tubing in stages using a tremie system, so the core is not damaged by hydraulic fracture.

20.4.8 Thermal monitoring of seepage

20.4.8.1 *General*

Temperature measurements can be used as an indirect means to determine the presence, location and quantity of seepage flows through embankment dams or in the foundation. In embankment dams and dikes, the internal temperature field is a function of the flow field. External temperature variations propagate within the dam body by means of conductive and convective heat transport processes, influencing the internal temperature distribution. This close interaction between the flow field and the temperature field allows using temperature as an easily measurable parameter to detect leakage or variations in the seepage pattern.

Leakage detection by means of temperature measurements have been typically implemented through three main approaches:

(a) Passive method, which employs natural temperature variations, e.g, annual variations or changes with time to detect and quantify anomalies in the flow field.
(b) Active method, which allows detecting the presence and movement of water by evaluating the thermal response after external heat is induced.
(c) Time lag method which can be used to estimate pore water velocities.

20.4.8.2 *Distributed fibre optic temperature sensing*

General characteristics

Distributed fibre optic temperature sensing (DTS) is useful if a large number of measuring points or an accurate representation of the temperature gradients is required. For distributed fibre optic temperature measurements a fibre optic cable is installed in a structure. This includes in standpipes within the embankment, along the downstream toe, or in the drainage system. Johansson and Sjodahl (2007) indicate that at that time 30 dams in Sweden had such installations.

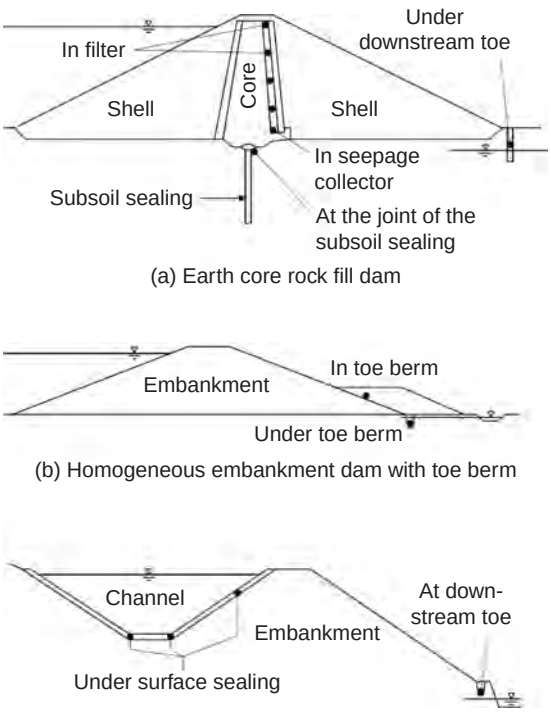


Figure 20.29 Examples of where fibre optic methods can be used to monitor seepage in dams and their foundations (Perzlmaier *et al.*, 2007).

The resulting measured temperature values have accuracy better than ±0.1°C and a spatial resolution of around 1.0 m. The measurement length of standard devices is up to 10 km (Johansson, 2006; Beck *et al.*, 2010). Since the measurement is carried out effectively at 1 m intervals along a cable the installation is the equivalent of many individual temperature probes.

ICOLD (2014) indicate that the optical fibre measuring system is insensitive to electromagnetic interference (lightning, stray electrical current, etc.). The measuring system requires external calibration of the temperature after the installation of the cable. The impulse transmitter/receiver can be easily replaced in case of damage or failure. The cable which is used as a sensor is integrated in the structure when installed and therefore well protected. It is not expected to fail under normal operating conditions.

Figure 20.29 shows examples of where fibre optic systems may be installed.

Seepage monitoring using passive fibre optic methods

The passive method relies on the seasonal variations in the reservoir water level and the inflow water temperature if horizontal temperature stratification in the reservoir

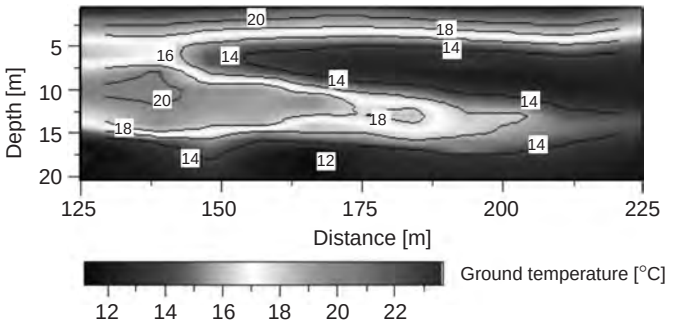


Figure 20.30 Example of temperature distribution from an array of temperature probes (Dornstadter and Heinemann, 2012).

occurs. The temperature field inside the dam will thus be a result of the seasonal temperature variations of the inflowing water and on the seepage flow rate passing through the dam. Johansson and Sjodahl (2007, 2009) give examples.

Seepage measurement using active fibre optic methods

In the active method heat-up method an electric current is applied to copper wires integrated in the fibre optic cable. Heat is applied in pulses or steadily and the rate of cooling of the soil used to assess the location of leakage or more permeable soil as evidenced by more rapid cooling. The method can be used to detect leakage through thin sealing elements such as geomembranes, asphaltic linings or concrete face slabs by placing the cable directly underneath them. Perzlmaier *et al.* (2007) describe the method in some detail. Dornstadter and Heinemann (2012) give examples. The point out how the lag time in changes in peaks and troughs in temperature in the reservoir and at the optic fibre measuring point can be used to estimate the pore water velocity.

20.4.8.3 *Thermotic sensors in stand pipes in the dam*

Dornstader (1996), Johansson (1996, 1997, 2007) and Dornstadter and Heinemann (2012) give examples and report that the method is quite successful for depths of up to 30–40 m and able to detect small variations in temperature. They indicate that average probe spacing is 20 m to detect temperature anomalies. A closer spacing (10 m) is required for lower permeability soils such as silt. Figure 20.30 shows an example of the output from a dike. The method has been used for 500 km of dams (to 2012).

The main disadvantage is that stand-pipes have to be inserted into the dam. They caution that measurements in open wells greater diameter than about 70 mm may be affected by convection currents and by raising and lowering instruments to take readings. To overcome this chains of temperature sensors at 1 m intervals are inserted into the tubes. Fibre optic methods can also be used.

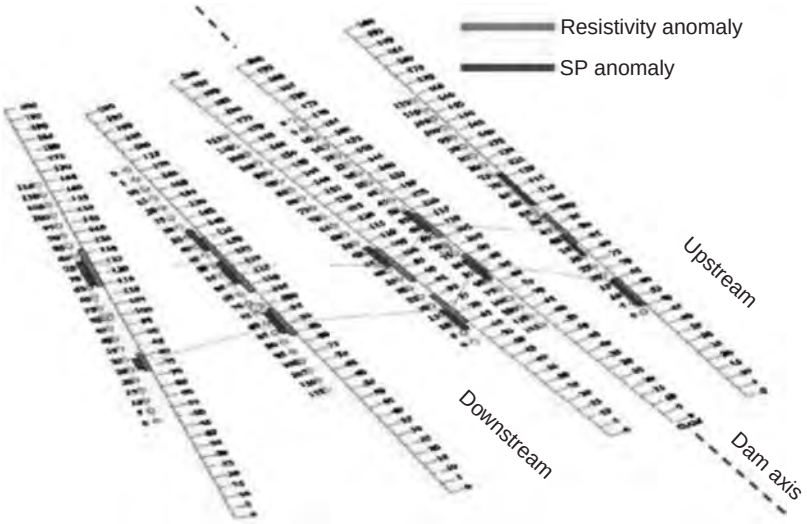


Figure 20.31 Presentation of Self Potential and resistivity anomalies along five lines at an embankment dam (ICOLD, 2014)

20.4.8.4 *Infra-red imaging of the downstream face of the dam and foundations*

Tedd and Hart (1988) report on trials on two dams. These showed that factors other than seepage also influence the temperature and the method would seem to be of limited usefulness. De Moel *et al.* (2010) trialled the method on several dams with some success.

20.4.9 Use of geophysical methods to detect seepage

There are a number of geophysical methods which can be used to detect seepage in embankment dams and their foundations. The following are brief descriptions of the methods and references for additional information.

20.4.9.1 *Self potential*

Self potential (also known as streaming potential), measurements are usually taken along several lines upstream on and downstream of the embankment. Figure 20.31 gives an example. References include Corwin (2005), Sheffer (2007) Revil and Boleve (2007), Boleve (2013) and ICOLD (2014). The authors have experience of the method successfully locating concentrated leaks in the floor of a reservoir.

20.4.9.2 *Resistivity*

Resistivity methods can be used to detect anomalies in the cores of dams by carrying out surveys along the dam crest. Figure 20.32 shows an example. The methods are also routinely used to detect zones of seepage in the foundations of dams on soil and rock.

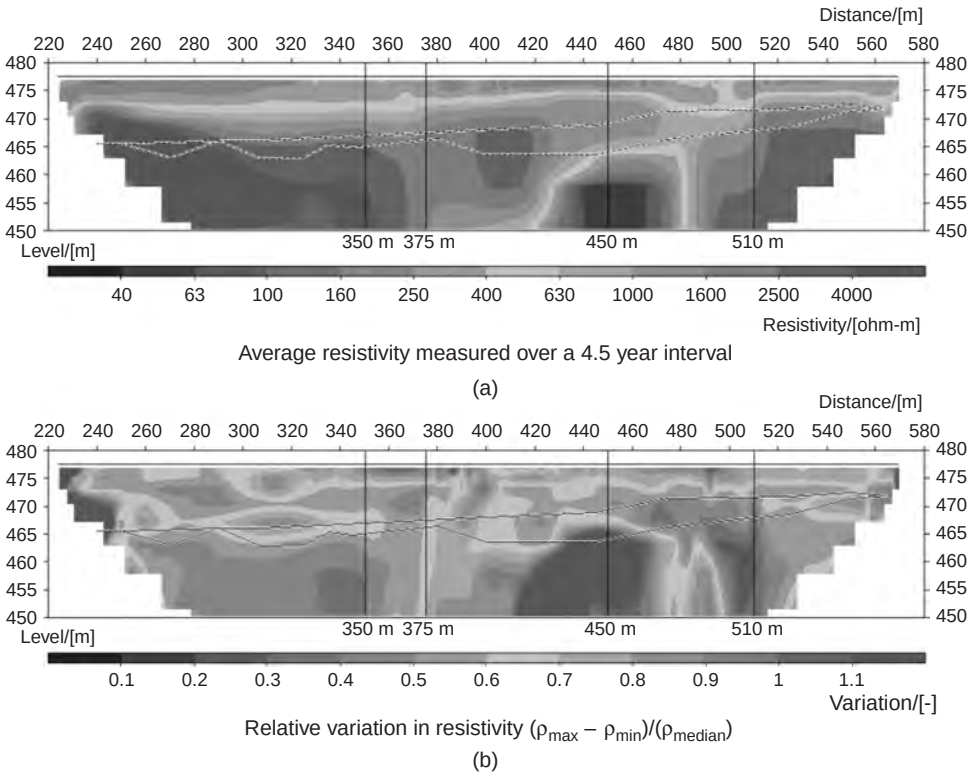


Figure 20.32 Measured resistivity distribution: (a) and relative variation; (b) along the longitudinal axis of Sadva earth and rockfill dam, Sweden (ICOLD, 2014).

References include Dahlin *et al.* (2008), Johansson and Sjodahl (2009), Sjodahl *et al.* (2007, 2011), Johansson (2007).

20.4.9.3 *Other methods*

Other methods which have been used to detect seepage include Ground penetrating radar (Blais *et al.*, 2007; ICOLD, 2014); electromagnetic profiling (Blais *et al.*, 2007); cross hole seismic (Garner and Vazinkhoo, 2007).

These are less commonly used than those described above.

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Appendix A

Methods for estimating the probability of failure by internal erosion and piping

PREAMBLE

The following information has been reproduced from

Fell, R., Foster, M., Davidson, R., Cyganiewicz, J., Sills, G. and Vroman, N. (2008). *A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping*. UNICIV Report R 446, The School of Civil and Environmental Engineering, University of New South Wales, Sydney, Australia 2052.ISBN:85841 413 9. www.engineering.unsw.edu.au/civil-engineering/uniciv-reports. This report is in two volumes, the Guidance Document, and the Supporting Document. Within this Appendix this is referred to as the "Piping Toolbox".

The information provided is to assist readers to estimate the conditional probability of initiation of erosion, and continuation given a particular reservoir level for a selection of failure modes.

How this information may be used is described in Chapter 8, Section 8.12.6.

If readers wish to carry out formal quantitative risk analyses they should refer to the original guidance document and the supporting document. Qualifications to the use of these are given in Section 8.12.6.4.

A1 Application of Tables for Estimating Conditional Probabilities

A1.1 GENERAL APPROACH

For each failure mode the conditional probabilities at each node in the event tree are estimated for each reservoir and earthquake load partition. They are used in the event trees to calculate probabilities of failure for each load partition. These probabilities are conditional on the reservoir or earthquake loading and are known as the "system response".

Sections A2 to A7 describe the Initiation of Erosion and Section A8 Continuation. Progression, Detection, Intervention and Repair, and Breach are not included here and reference must be made to the Piping Toolbox.

Tables are presented within the sections to give guidance on the estimation of conditional probabilities. These tables have been developed to model the physical processes so far as practical. The probabilities have been assessed using the expert judgment of the authors of the Piping Toolbox documents. Where practical these have been anchored to historic data. This has mainly been possible in the estimation of the probability of initiation of erosion in concentrated leaks and is discussed further in Section A1.2 and in the Supporting Document.

A1.2 HISTORICAL FREQUENCIES OF CRACKS AND POORLY COMPACTED OR HIGH PERMEABILITY ZONES IN EMBANKMENTS

Concentrated leak erosion may occur in cracks caused by differential settlement, desiccation, or in poorly compacted zones. In poorly compacted zones initiation of erosion may be a result of the voids between aggregated soil particles giving higher permeability and continuous open paths, or collapse settlement of the poorly compacted layer leading to a flaw or continuous open path in which water can flow and erode the sides of the flaw as happens in a crack.

Estimated historical frequencies of the occurrence of cracking, hydraulic fracture or poorly compacted or high permeability zones in embankments are presented in Table A1.1. They have been determined from analysis of historic dam accidents and failures, allowing for under reporting of the incidents in the database, cracking which may have been present in the dams in the database but were sufficiently resistant to erosion for erosion not to initiate, the cracks sealed by swelling, were above the Pool of Record (POR) of the dams, or erosion was stopped by filters. Details of the database and the analysis are given in the Supporting Document.

The frequencies in Table A1.1 are for a crack, hydraulic fracture or poorly compacted or high permeability zone being present, and do not include the assessment of whether erosion initiates in the crack, hydraulic fracture or poorly compacted or high permeability zone. These historic frequencies have been used as a basis for anchoring the estimated probabilities of a flaw being present in which erosion may initiate.

When applying the historic frequencies to predict future behavior it is assumed that the "Reservoir Level above Pool of Record" values apply when the reservoir level exceed the pool of record level by at least 1 foot (0.3 metres). This is so the above "pool of record" frequencies apply when the reservoir is testing significant new areas of the dam.

The historical frequencies for cracking or poorly compacted or high permeability zone in the dam body were further subdivided into the various mechanisms of crack formation. This was necessary so as to avoid the double counting of the historical frequencies when the probabilities for each initiating mechanism are added together. Table A1.2 presents the estimated historical frequencies for each of the mechanisms.

As described in the Supporting Document, the historical frequencies presented in Table A1.1 and Table A1.2 represent the average frequencies for cracks/ hydraulic fracture/poorly compacted or high permeability zones across the population of dams

Table A1.1 Estimated historical frequencies of cracking, hydraulic fracture or poorly compacted or high permeability zones in embankment dams.

	Estimated historical frequencies of cracking, hydraulic fracture or poorly compacted or high permeability zones		
Location of cracking/Poorly compacted or high permeability Zone	*First filling*	*Reservoir Level above Pool of Record*	*Pool of Record Reservoir Level below*
In embankment (dam body)	0.014	0.014	0.001 per annum
Associated with conduit	0.01	0.01	0.0007 per annum
Associated with concrete wall or structure through embankment	0.004	0.004	0.0003 per annum

Table A1.2 Historical frequencies for cracking or poorly compacted zone in the embankment dam body.

	Estimated historical frequencies of cracking, hydraulic fracture or poorly compacted or high permeability zones		
Mechanism for Cracking or Poorly Compacted or high permeability Zone	*Proportion of Cases*	*Reservoir Level above Pool of Record*	*Reservoir Level below Pool of Record (per annum)*
All incidents (cracking, hydraulic fracture and poorly compacted or high permeability zone)	100%	0.014	0.001
Cracking and hydraulic fracture	63%		
In upper part	(47%)	0.007	0.0005
In middle/lower part	(16%)	0.002	0.0002
Poorly compacted or high permeability zone in upper and lower parts (total)	37%	0.005	0.0004

in the database. These represent a large number of dams of varying age and varying levels of engineering design and construction practice. They do not represent the historical frequencies for the "average dam" a term which has been applied previously (e.g. Fell et al 2003, 2004). It would be expected that the portfolio of well engineered Reclamation, USACE and Australian dams would have average probabilities estimated based on these historical data less than the average historical frequencies presented in the tables.

The historical frequencies for the "Below Pool of Record" case are quoted as annualized frequencies. For ease of application these are applied directly as conditional probabilities as the majority of dams have a seasonal cycle of reservoir fluctuation and reach a particular reservoir level each year.

The database of incidents is relatively small and because of this there should be no further subdivision of the failure and accident statistics.

A1.3 HISTORICAL FREQUENCIES FOR INTERNAL EROSION IN AND INTO THE FOUNDATION

The conditions in the foundations of dams are inherently more complex and varied than in the dam body, and hence historic frequencies for internal erosion in and into the foundation are more difficult to interpret and apply. One of the key issues for these modes of internal erosion is whether continuous seepage paths or open defects are present, and historic performance data provides very little information to aid in the assessment. For this reason, the use of historic performance data for anchoring the conditional probabilities for internal erosion through the foundation and internal erosion from the embankment into the foundation is not recommended.

A1.4 ESTIMATING CONDITIONAL PROBABILITIES

A1.4.1 Estimating conditional probabilities using relative importance factors and likelihood factors

Most of the Sections for estimation of conditional probabilities are structured so there is a Table of Factors affecting the likelihood. These tables are structured to show:

- The Factors
- The Relative Importance of this Factor (RF) with numeric weightings (usually three Factors with Relative Importance weightings of 3, 2 and 1).
- Likelihood Factors (LF) for which there are descriptions. Generally there are four for each Factor, with Likelihood weightings of 4, 3, 2 and 1.

There is then a second table which links $\sum (\text{RF}) \times (\text{LF})$ to the conditional probability. Where there is historic data to "anchor" the probabilities these are shown; e.g. [0.0005] for Table A2.2, "Below POR". The tables have two or more sets of probabilities. Where the "anchor" probabilities are estimated by expert judgment of the Toolbox development team they are shown with rounded brackets, e.g. (0.003). The conditional probabilities on these tables are on a log scale, and interpolation between the bracketed probability values should be based on log interpolation.

The "Below POR" figures are for reservoir level stages with a representative level up to and 1 foot (0.3 m) above the historic high reservoir pool level (POR). The "Above POR" figures are for representative reservoir level stages at least 1 foot (0.3 m) above this historic high reservoir pool level.

These anchor probabilities are a form of a "base rate frequency" and the approach used in the tables is a base rate frequency approach.

Those carrying out the risk analysis are required to choose which of the descriptors for each Factor best reflects conditions at the dam. Where there is little data this assessment should be made on the best available information and using judgment based on geological conditions and experience elsewhere on dams of similar age and design.

For some conditional probability estimates it has been necessary to modify the format of the table to better model Relative Importance or Likelihood distributions.

In the tables the term "negligible" means that the contribution to the probability of failure would be very small indeed, insufficient to affect the outcome.

A1.4.2 Estimating conditional probabilities using scenario tables

Where the number of factors affecting the estimation of conditional probabilities is few or to be based on limited data, "Scenario Tables" are used. These have been developed by the Toolbox development team based on published information, and the experience of the team members.

These tables present ranges of conditional probabilities within which the risk analysis team are to select their best estimate based on the details of the dam they are analyzing.

A1.4.3 Estimating conditional probabilities using probability estimate tables

In some Sections, e.g. those describing the assessment of the probability of initiation of erosion in a crack, (Section A2.4.2) and initiation and progression of backward erosion in cohesionless soils (Section A3.6.2), the Toolbox development team have carried out analyses to simplify the estimation of conditional probabilities from input data which itself has significant uncertainty, and the analysis methods themselves have uncertain outputs.

The assumptions made to develop these tables are described in the Supporting Document.

A1.5 LENGTH EFFECTS

The effect of the length of the embankment being considered may have an influence on the assessed probability of internal erosion and piping. The effect is dependent on the failure mode, how the embankment is partitioned for the analysis, and how the conditional probabilities are assessed.

Many failure modes are independent of length because they are related to specific features in the embankment, such as a conduit, contact with a wall, and differential settlement of a major change in foundation profile. In many embankments, these failures modes contribute most to the likelihood of internal erosion and piping.

The failure modes which are potentially affected by length are cracking due to desiccation (either by drying and/or freezing), high permeability zones in the embankment (e.g. due to poorly compacted layers in the core), high permeability layer on the core-foundation contact, internal erosion of the embankment into or at a rock or soil foundation, and backward erosion in cohesionless soils in the foundations of dams. For these failure modes it is the likelihood of initiation of erosion for which length may

be a factor. The length can be accounted for by considering it in determination of the probability; e.g. "what is the probability of a through going crack in the core due to desiccation in this 1000 feet length of embankment which so far as can be ascertained all has the same geometry, zoning and material properties?"

In statistical terms if all sections of the embankment are exactly the same, then they are perfectly correlated and the probability of at least one crack in the whole of the embankment is the same as for one section regardless of length. If each section is completely independent of the others, i.e. different construction materials, different specifications and construction methods, different cover (e.g. road pavement) over the core, then each section should be considered separately and the probabilities added using De Morgan's rule;

$$P = 1 - (1 - P_1) \times (1 - P_2) \times (1 - P_3) \ldots \textit{ etc.}$$

More information is provided in the Piping Toolbox.

A1.6 NATURE OF THE ESTIMATES OF PROBABILITIES GIVEN BY THE TOOLBOX

A1.6.1 The toolbox gives "Best Estimate" probabilities

The methods in the Toolbox provide "best estimates" of the conditional probabilities and hence "best estimate" probabilities of failure.

The estimates are determined by expert judgment based on analyses and laboratory tests modeling the physical processes. They are designed to avoid systematic bias towards conservative or non-conservative probabilities.

Probabilities for some of the most important initiating modes within the embankment are calibrated against historic performance of dams from a large database of around 10,000 dams in the ICOLD (1986) survey of failures and accidents. This is discussed in detail in Section A1.2. Where this has not been possible expert judgment of the team developing the Toolbox based on extensive experience in dams and risk assessment has been used, taking into account the feedback from trials and reviews of the Toolbox in Reclamation and USACE.

The methods in the Toolbox are likely to be more reliable in assessing relative probabilities between failure modes and between dams than assessing absolute values.

A1.6.2 Adjusting the Toolbox Best Estimates

See Section A4.6 of the Piping Toolbox

A1.6.3 Limitations of the Methods used in the Toolbox

The Toolbox is based on the (2007) state of the art on modeling the mechanics of initiation of internal erosion in cracks and other flaws, by backward erosion and suffusion. The methods available are sufficient to form the basis of the Toolbox, and

are to be preferred to methods based only on historic data, or expert judgment anchored on historic data such as those detailed in Fell *et al.* (2003, 2004).

The mechanics of continuation (filter action) are fundamentally simpler, more extensively researched and the methods are less subjective than for initiation, provided the data to do the analyses is available.

The modeling of progression, detection and intervention, and breach are more subjective and largely based on case studies and expert judgment.

The methods used here are a significant improvement on the methods described in Fell *et al.* (2003, 2004) which have been used in Australia and as input to the Reclamation methods.

It is recommended that those using Fell *et al.* (2003, 2004) now use the methods described in this Guidance Report.

Most of the logic, modeling, analysis, laboratory testing, expert elicitation techniques used in this document continues to be actively researched and/or studied. Case histories continue to occur that give more insight to the process involved. It is likely that these developments in understanding will result in improved methods for assessing probabilities of initiation, continuation, and progression of internal erosion, and breach mechanics. When this occurs it will be necessary to revise the Toolbox.

A1.6.4 Assessment of probabilities of failure for failure modes which are not covered by the toolbox

In some dams there may be failure modes which are not well modeled by the Toolbox. For these failure modes the risk analysis team should develop an event tree to model the failure mode. Conditional probabilities within the event tree should be estimated using the Toolbox where the nodes are common to the Toolbox event tree, and by expert judgment for the other nodes. Appendix E of the Piping Toolbox details how this should be done.

A1.7 MODELING UNCERTAINTY IN THE ESTIMATES OF CONDITIONAL PROBABILITIES

See Section A4.7 of Piping Toolbox

A1.8 SUMMARIZING (MAKING THE CASE)

See Section A4.8 of Piping Toolbox

A1.9 COMBINING PROBABILITIES

See Section A4.9 of Piping Toolbox

A2 Probability of Initiation of Erosion in Transverse Cracks in the Embankment

A2.1 OVERALL APPROACH

a) Estimate the probability of a transverse crack for normal and hydrologic loading for each of the initiating mechanisms that can lead to cracking and low stress zones in which hydraulic fracture can occur. These are:

- Upper parts of the embankment (Section A2.2)
- Differential settlement cross valley (IM1)
- Differential settlement adjacent to a cliff (IM2)
- Differential settlement cross valley arching (IM3)
- Differential settlement as a result of cross section settlement (IM4)
- Differential settlement in the foundation beneath the core (IM5)
- Differential settlements due to embankment staging (IM6)
- Desiccation cracking (IM7, IM8)

Lower and middle parts of the embankment (Section A2.3)

- Differential settlement cross valley (IM9)
- Differential settlement as a result of cross section settlement due to arching (IM10)

Table A2.1 Factors influencing the likelihood of cracking or hydraulic fracturing in the upper part of embankment dams – Cross Valley Differential Settlement (IM1).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Cross valley profile under embankment core[1]	(3)	Uniform abutment profile without benches Narrow bench very low in the abutment $b/h_2 < 0.5$, $h_2/h_1 > 1.5$ Refer to Note (a)	Wide bench low in the abutment $b/h_2 > 1; h_2/h_1 > 1$	Wide bench in upper half to one third of the abutment $b/h_2 > 1; 0.5 < h_2/h_1 < 1$ or narrow bench in upper half to one third of the abutment, $b/h_2 > 0.5; h_2/h_1 < 0.25$	Wide bench near the crest in the abutment $b/h_2 > 1$ $0 < h_2/h_1 < 0.5$
Slope of abutments under embankment core[1]	(2)	Gentle abutment slope $\beta_1 < 30°$	Moderate abutment slopes $30° < \beta_1 < 45°$	Steep abutments $45° < \beta_1 < 60°$	Very steep abutments, $\beta_1 > 60°$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m. Refer to Note (b)

Notes: (a) See Figure A2.1 for definitions of b, h_1, h_2, β.
(b) For dams higher than 400 ft (120 m) assign a likelihood factor of 5.
(c) The method of applying this type of table is described in Section A1.4.

- Differential settlement due to soil in the foundations (IM11)
- Differential settlement due to small scale irregularities in the foundation profile (IM12)

This is done using Tables A2.1 to A2.15 in Section A2.2 for cracking in the upper part of the dam, and Tables A2.16 to A2.21 in Section A2.3 for cracking in the middle and lower parts. The embankment is split into the upper and middle and lower parts to take into account the likely locations for each of the crack initiating mechanisms. The inputs to these tables are modified by observed settlements or cracking using Tables A2.22 and A2.23 in Section A2.3.5.

For most dams not all mechanisms will be present and those mechanisms are assigned a zero probability. The details for screening the mechanisms are described in Section A3.4 of the Piping Toolbox.

Table A2.2 Probability of a cracking or hydraulic fracture in the upper part of embankment dams-Cross Valley Differential Settlement versus ∑(Relative importance factor (RF) x (Likelihood factor (LF)).

0.00001	0.00005	0.00015	[0.0005]	0.005	0.02	Below POR
0.0001	0.0005	0.002	[0.007]	0.05	0.2	Above POR
6	9	11	13	18	24	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

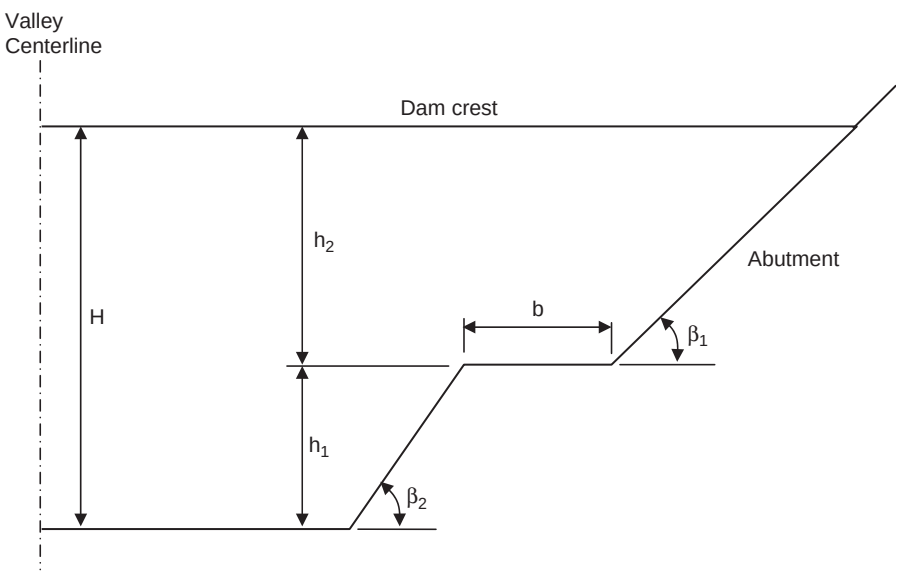


Figure A2.1 Definition of terms used to describe cross valley geometry.

b) Estimate the maximum likely crack width at the surface of the core for each of the initiating mechanisms which apply as described in Section A2.4. Then estimate the likely crack width at the reservoir level being considered.
c) Estimate the probability of initiation of erosion for this mechanism given this estimated crack width, seepage gradient across the core, and the properties of the soil in the core (P_{IC}) using Section A2.4.
d) Estimate the probability of initiation of erosion $(P_I) = (P_C) \times (P_{IC})$ for each initiating mechanism. The probability estimates for initiation of erosion for each initiating mechanism are not added together, but are carried through the event trees for each failure path.

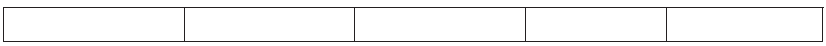
Table A2.3 Factors influencing the likelihood of cracking or hydraulic fracturing in the upper part of embankment dams-Differential settlement adjacent a cliff at the top of the embankment (IM2).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Cross valley profile under embankment core[a][b]	(3)	Wide bench $W_b/H_w > 2.5$ Note. If this condition is present, probability = zero	Bench adjacent to cliff $1.0 < W_b/H_w < 2.5$	Narrow bench adjacent to cliff $0.25 < W_b/H_w < 1.0$	No or very narrow bench adjacent to cliff. $W_b/H_w < 0.25$
Slope of abutments under embankment core[a]	(2)	Gentle abutment slope $\beta_1 < 25°$ Note. If this condition is present, probability = zero	Moderate abutment slopes $25° < \beta_1 < 45°$	Steep abutments $45° < \beta_1 < 60°$	Very steep abutments, $\beta_1 > 60°$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Note: (a) See Figure A2.2 for definitions of W_b, H_w, β_1
(b) This mechanism only applies for $W_b/H_w < 2.5$.

Table A2.4 Probability of a cracking or hydraulic fracture in the upper part of embankment-Differential settlement adjacent a cliff at the top of the embankment versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).

negligible	negligible	negligible	negligible	[0.0005]	0.002	0.02	Below POR
negligible	negligible	negligible	negligible	[0.005]	0.02	0.2	Above POR



6 9 11 13 14 19 24 RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

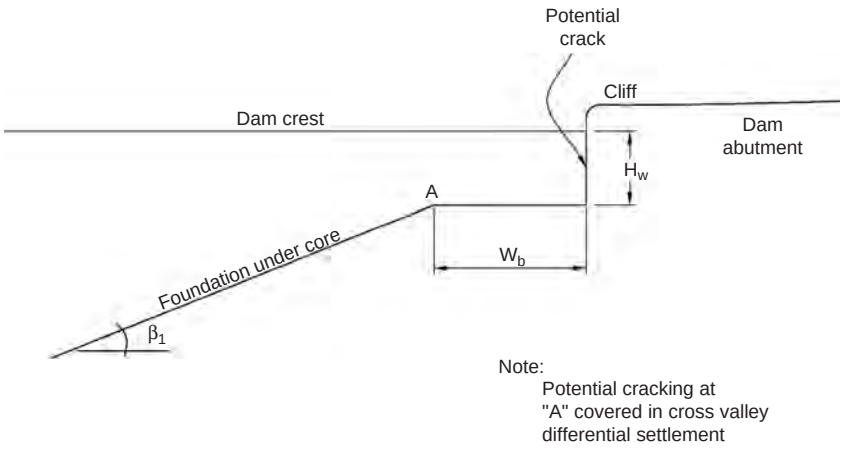


Figure A2.2 Cracking or hydraulic fracture adjacent cliffs due to differential settlement of the embankment. Note that this mechanism only applies for $W_b/H_w < 2.5$.

Table A2.5 Factors influencing the likelihood of cracking or hydraulic fracturing in embankment due to cross valley arching (IM3).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Slope of abutments under embankment core[a]	(3)	Moderate abutment slope $\beta_1, \beta_2 < 45°$	Moderate steep abutment slopes $45° < \beta_2, \beta_2 < 60°$	Steep abutments $60° < \beta_1, \beta_2 < 75°$	Very steep abutments, $\beta_1, \beta_2 > 75°$ β_1 near vertical, β_2 60°
Cross valley geometry under embankment core[a]	(2)	$W_v/H > 0.75$ Note. If $W_v/H > 2$, Exclude this failure mode	$0.4 < W_v/H < 0.75$	Narrow deep valley $0.25 < W_v/H < 0.4$	Very narrow deep valley $W_v/H < 0.25$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Note: (a) See Figure A2.3 for definitions of W_v, H, β_1 and β_2.
(b) If the soil in the lower part of the core is poorly compacted or subject to collapse compression on saturation, and the upper part is not, increase weighted factor (LF x RF) by 1 or 2.

Table A2.6 Probability of cracking or hydraulic fracturing in embankment due to cross valley arching versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).

negligible	negligible	0.00005	0.0001	0.0005	0.004	0.02 Below POR
negligible	negligible	0.0005	0.001	0.007	0.05	0.2 Above POR
6	9	10	13	17	21	24 RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

A2.2 ESTIMATING THE PROBABILITY OF TRANSVERSE CRACKING (P_C) IN THE UPPER PART OF THE DAM

A2.2.1 Likelihood of a transverse crack due to cross valley differential settlement (IM1)

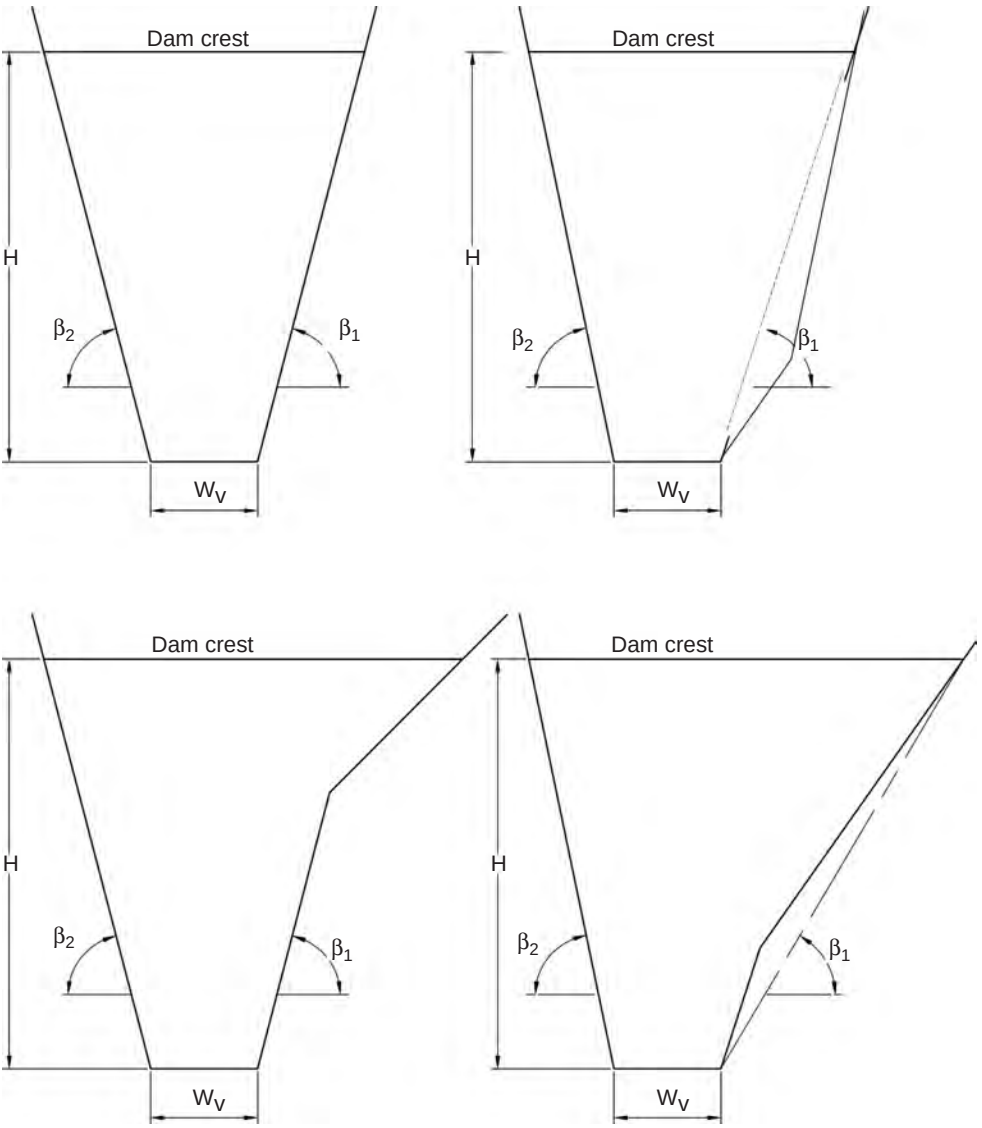


Figure A2.3 Longitudinal profiles of the dam showing the definition of terms for cross valley arching.

A2.2.2 Likelihood of transverse cracking resultant on cross section settlement due to poorly compacted shoulders (IM4)

A2.2.3 Likelihood of transverse cracking due to differential settlements in soil in the foundation beneath the core (IM5)

Table A2.7 Factors influencing the likelihood of cracking or hydraulic fracturing in the upper part of embankment – cross section settlement resulting from poorly compacted shoulders (IM4).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
EITHER Embankment zoning and Compaction of outer zone Excluding sloping core earth and rockfill dams	(3)	Zoned earthfill, earthfill with filter drains or homogeneous (all materials similar modulus) Note: if have evidence from relative settlements of core and shoulders that the materials have a similar modulus, then probability =0	Zoned earthfill, earthfill with filter drains, modulus of outer zones lower than core. Central core earth and rockfill (or "gravel fill"), with well compacted shoulders and core.	Central core earth and rockfill (or "gravel fill"), rockfill or gravel" fill compacted by dozer tracking or by small rollers in thick layers	Central core earth and rockfill, uncompacted (dumped) rockfill
OR Embankment zoning and Compaction of outer zone, sloping core earth and rockfill dams	(3)	Core sloped steeper than 45° but within limits of sloping core embankment. Rockfill (or "gravel fill"), and core .well compacted, and with similar moduli.	Core sloped flatter than 45° Rockfill (or "gravel fill"), and core .well compacted.	Core sloped flatter than 45°. Rockfill (or "gravel fill"), rockfill or gravel" fill compacted by dozer tracking or by small rollers in thick layers	Core sloped flatter than 45°. Construction staged with rockfill in the lower part of the dam compacted to a higher modulus than the upper part.[a] OR uncompacted (dumped) rockfill
Core geometry Width (W)/ Height (H)[b]	(2)	W/H > 3	1.5 < W/H < 3	0.5 < W/H < 1.5	W/H < 0.5
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Note. (a) If there is a large difference in moduli, a probability of cracking between 0.02 and 0.2 may be applied.
(b) See Figure A2.4 for definition of width W and Height H. Width should be taken at the base of the core.

Table A2.8 Probability of a transverse crack or hydraulic fracture resultant on cross section settlement resulting from poorly compacted shoulders versus ∑(Relative importance factor (RF)) x (Likelihood factor (LF)).

0.00001	0.00002	0.00005	0.0002	[0.0005]	0.002 Below POR
0.0001	0.0002	0.0005	0.002	[0.007]	0.02(1) Above POR
6	9	11	13	18	24 RF x LF

Note: (1) If there is a large difference in modulus between the core and shoulders, a probability of cracking between 0.02 and 0.2 may be applied.
Note: "POR" refers to the Pool of Record level + 0.3 m.

A2.2.4 Likelihood of transverse cracking due to differential settlements due to embankment staging (IM6)

This situation arises where the embankment has been staged during construction. There is a potential for differential settlement to occur if the "existing" (first stage) embankment is a significantly higher modulus than the remainder of the embankment.

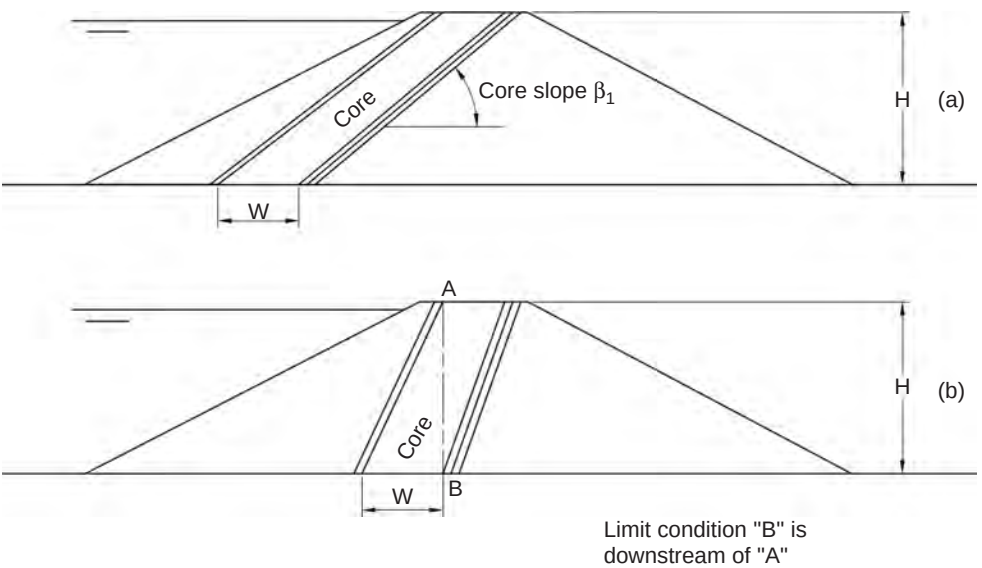


Figure A2.4 Sloping core dam (a) Definitions of terms. (b) Limit of what constitutes a sloping core dam.

If this is the situation, use the method described in Section A3 to assess the likelihood a crack or hydraulic fracture will result. If however there is no or little difference in the modulus, this mode may be ignored. In most cases the latter will apply.

Table A2.9 Factors influencing the likelihood of cracking or hydraulic fracturing in the upper part of embankment dams-settlement resulting from differential settlements in soil in the foundation (IM5).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Foundation geology and geometry[c]	(3)	Rock foundations or uniform soil foundations[a]	Shallow soils or soils with gradual variation in depth and compressibility sufficient to cause differential settlement of less than 0.2% of the embankment height	Moderate depth of compressible soil in the foundation sufficient to cause differential settlement of 0.2% to 0.5% of the embankment height	Deep compressible soil in the foundation[b] sufficient to cause differential settlement of $>0.5\%$ of embankment height
Slope of the sides of the compressible zones[d]	(2)	Gentle $\alpha < 30^\circ$	Moderate $30^\circ < \alpha < 45^\circ$	Steep $45^\circ < \alpha < 60^\circ$	Very steep $\alpha > 60^\circ$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m high	Very high dams >60 m high

Notes: (a) If there is no compressible soil in the foundation this mode does not apply.
(b) Including soils which collapse on saturation and which have not been treated or removed during construction.
(c) See Figure A2.5 for typical scenarios which may lead to differential settlement.
(d) See Figure A2.5 for definition of slope α.

Table A2.10 Probability of a crack or hydraulic fracture due to differential settlement in the foundation versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).

negligible	negligible	0.00005	0.0002	[0.0005]	0.003	0.02	Below POR
negligible	negligible	0.0005	0.002	[0.007]	0.03	0.2	Above POR



6	8	9	11	13	18	24	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.
Note: (1) This mode does not apply if there is no compressible soil in the foundation beneath the core.

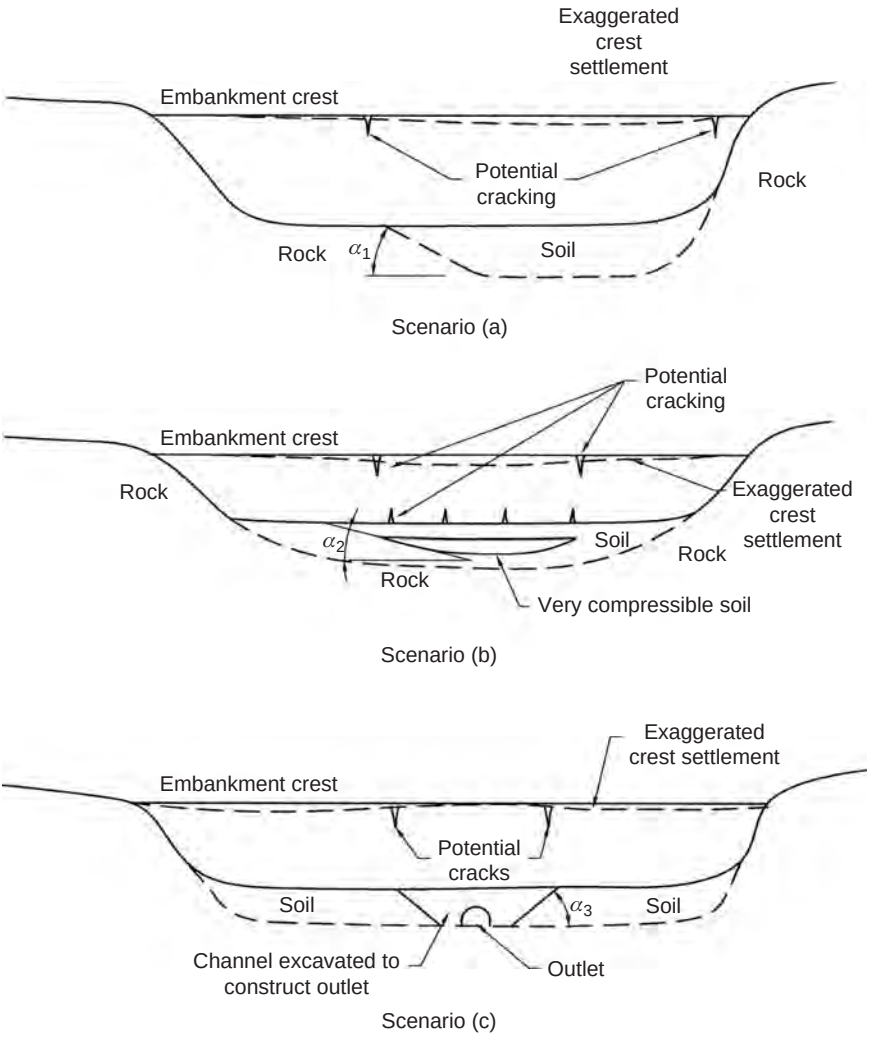


Figure A2.5 Typical scenarios which may lead to differential settlement in the foundation.

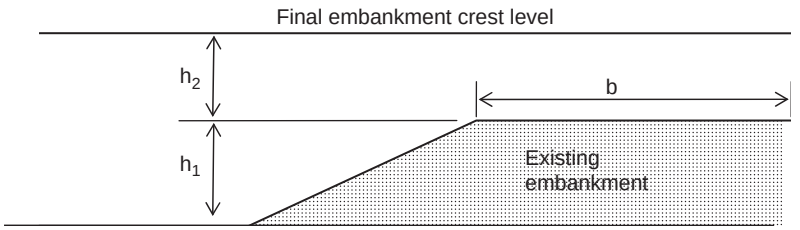


Figure A2.6 Longitudinal section through staged embankment.

A2.2.5 Likelihood of transverse cracking due to desiccation (IM7, IM8)

Desiccation in the crest of the dam (IM7)

FIRST Consider the maximum likely depth of desiccation cracking for the soil in the core of the dam, and the climate using Table A2.13. If the reservoir stage being considered is below the likely depth of desiccation cracking, the probability of a crack due to desiccation cracking can be assumed to be zero.

SECOND For cases where the reservoir stage is above the base of potential desiccation cracking, follow the procedure below.

Table A2.11 Factors influencing the likelihood of cracking in the upper part of embankment dams-cracking in the crest due to desiccation by drying (IM7).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Crest zoning and surface layer over core	(3)	Road pavement cover[(a)] with base layer 300 mm or more thick, and/or Rock fill or non plastic granular layer at least 1 metre thick	No road pavement cover[(a)], with non-plastic granular material, 150 mm to 300 mm thick.	Thin (less than 75 mm) surface gravel layer with no pavement cover[(a)] or Low plasticity granular transition layer over core	No surface layer. Dam core extends to crest level
Climate	(2)	Temperate climate, uniform rainfall throughout the year	Seasonal climate with annual rainfall greater than 500 mm and no prolonged hot dry periods	Monsoonal or other distinct wet and dry periods in the year. Summer maximum temperatures $>30°C$	Arid climate, less than 250 mm rainfall, High summer temperatures
Plasticity of core material	(1)	Low plasticity to non plastic	Medium to low plasticity	Medium to high plasticity	High plasticity

Note: (a) Road pavement cover may comprise concrete, asphalt or bitumen seal.

Table A2.12 Probability of a transverse crack, cracking in the crest due to desiccation by drying versus $\sum$(Relative importance factor (RF)) x (Likelihood factor(LF)).

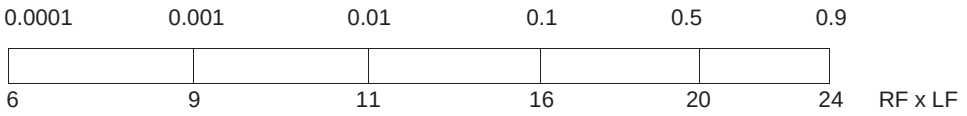


Table A2.13 Screening Tool. Maximum likely depth of desiccation cracking for a gravel surface layer with no road pavement cover based on climate.

Climate	*Maximum likely depth with gravel layer with no road pavement cover*[a] *(metres)*	*Maximum likely depth with no surface layer (dam core extends to crest level) (metres)*
Arid climate, less than 250 mm rainfall, high summer temperatures	4.5	7
Monsoonal or other distinct wet and dry periods in the year. Summer maximum temperatures >30°C	4	6
Seasonal climate with annual rainfall greater than 500 mm and no prolonged hot dry periods	3	5
Temperate climate, uniform rainfall throughout the year	2	4.5

Note: (a) Road pavement cover may comprise concrete, asphalt or bitumen seal.

Table A2.14 Factors influencing the likelihood of cracking on seasonal shutdown layers during construction and staged construction surfaces due to desiccation by drying (IM8).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Construction practices regarding clean-up of desiccated layers after construction shutdowns or the surface of the earlier stage of the dam	(4)	Very good control and clean-up practices. Desiccated layers removed from embankment and replaced with new soil or adequately reworked to specified moisture content. If this condition is present, probability = 0, regardless of the other factors	Good control and practices, surfaces scarified, moisture adjusted to specified range, surface re-compacted.	Moderate control. Attempts to scarify desiccated layers, but depth of scarifying insufficient or difficulties with moisture control	Poor control. No attempt to scarify or remove desiccated layers, poor moisture control practices
Climate	(2)	Temperate climate, uniform rainfall throughout the year	Seasonal climate with annual rainfall greater than 500 mm and no prolonged hot dry periods	Monsoonal or other distinct wet and dry periods in the year. Summer maximum temperatures >30°C	Arid climate, less than 250 mm rainfall, High summer temperatures
Plasticity of core material	(1)	Low plasticity to non plastic (LL < 20%)	Medium to low plasticity (20% < LL < 40%)	Medium to high plasticity (40% < LL < 50%)	High plasticity (LL > 50%)

Desiccation cracking on seasonal shutdown layers and on the surface of staged embankments (IM8)

FIRST (a) This mechanism only applies above the level of saturation of the core. Below that any desiccation cracks should have swelled and closed. If the seasonal shutdown layer is below the Pool of Record, use the "Below POR" probabilities in Table A2.15.

(b) This mechanism only applies where there has been a seasonal shutdown during construction, or the embankment has been staged. The descriptions in Table A2.14 are to be assessed according to the conditions across the width of the core.

SECOND Where the mechanism applies, estimate the probability of desiccation cracking using Table A2.14 and Table A2.15, and multiply this probability by the assessed likelihood the layer will be continuous across the core.

Table A2.15 Probability of a transverse crack on seasonal shutdown layers during construction and staged construction surfaces due to desiccation by drying versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).

negligible	negligible	negligible 0.0001	0.001	0.01	0.1	Below POR
negligible	negligible	negligible 0.001	0.01	0.1	0.9	Above POR
7	10	12	13	18	22	28 RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

A2.3 ESTIMATING THE PROBABILITY OF TRANSVERSE CRACKING OR HYDRAULIC FRACTURE (P_C) IN THE MIDDLE AND LOWER PARTS OF THE DAM

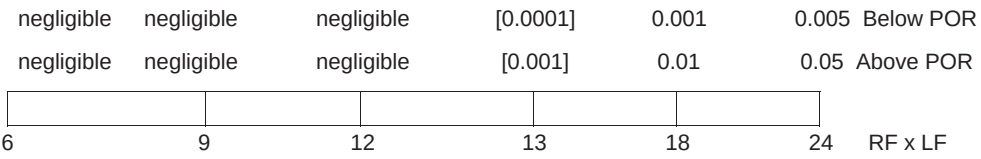
A2.3.1 Likelihood of transverse cracking or hydraulic fracture due to cross valley differential settlement (IM9)

Table A2.16 Factors influencing the likelihood of cracking or hydraulic fracture in the middle and lower parts of embankment dams-cross valley differential settlements (IM9).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Cross valley profile under embankment core	(3)	Uniform abutment profile without benches	Minor bench in lower half of the abutment, not persistent across the core	Bench in lower half of the abutment $b/h_2 > 0.5$; 1.5 $< h_2/h_1 < 2$	Wide bench in lower half of the abutment $b/h_2 > 1$; 1 $< h_2/h_1 < 1.5$
Slope of the abutments under embankment core	(2)	Gentle abutment slope, $\beta_2 < 30°$	Moderately abutment slopes, $30° < \beta_2 < 45°$	Steep abutments, $45° < \beta_2 < 60°$	Very steep abutments, $\beta_2 > 60°$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Note: See Figure A2.1 for definitions of b, h_1, h_2, β_2.

Table A2.17 Probability of a transverse crack or hydraulic fracture in the middle and lower parts of embankment dams due to cross valley differential settlements versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).



Note: "POR" refers to the Pool of Record level + 0.3 m.

A2.3.2 Likelihood of transverse cracking or hydraulic fracture due to differential settlement causing arching of the core onto the shoulders of the embankment (IM10)

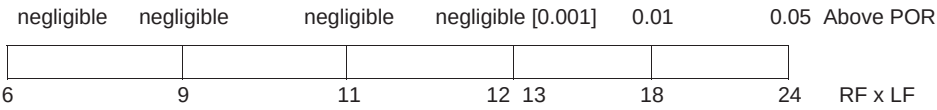
This mode is applicable to central core earth and rockfill (or gravel shells) dams and puddle core earthfill dams. It is not applicable to all other the following dam types (including dams with a sloping core).

Table A2.18 Factors influencing the likelihood of cracking or hydraulic fracturing in the middle and lower parts of embankments – settlement resulting from arching of the core onto the shoulders (IM10).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Core geometry Width (W)/ Height (H)	(3)	W/H > 1.0[c]	0.5 < W/H < 1.0	Narrow core, 0.25 < W/H < 0.5	Very narrow core, W/H < 0.25
Relative stiffness of core and shells	(2)	Core has higher modulus than shells. Shoulders poorly compacted or dumped. Core compacted >98% SMDD[c]	Modulus of core same or marginally lower than shoulders. Shoulders well compacted and high modulus. Core compacted to >98% SMDD at a moisture content between −2% and +1% of standard OWC	Core lower modulus than outer stiffness. Shoulders well compacted and high modulus. Either core compacted 0% to 2% wet of OWC and between 95% and 98% SMDD or 2% to 3% dry of OWC and <95% SMDD[b]	Core much lower modulus than outer shoulders or subject to collapse compression. Shoulders well compacted and high modulus Either core compacted >2% wet of OWC, or more than 3% dry of OWC and <95% SMDD [b]
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Notes: (a) The most likely location for arching to occur is in the upper to middle part of the dam.
(b) In core materials compacted dry of optimum moisture and to a density ratio less than about 95%, collapse compression of the core may occur. This may lead to arching and low stresses. It may also lead to softened zones and even a crack in the vicinity of the contact between the saturated and unsaturated parts of the core, i.e. at the phreatic surface.
(c) The likelihood of arching is negligible for embankments with these characteristics.
(d) The mechanism is considered to be significant only for reservoir levels above the pool of record.
(e) W = width of the core at depth H below the crest of the embankment.
(f) Finite element analyses which properly model the history of the dam, and its properties including collapse on saturation may be used to assess the likelihood of cracking or hydraulic fracture.

Table A2.19 Probability of a transverse crack or hydraulic fracture in the middle and lower parts of embankment dams – settlement resulting from arching of the core onto the shoulders of the embankment versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).



Note: "POR" refers to the Pool of Record level +0.3 m. The probability for this initiating mechanism is assumed to be negligible for the Below POR cases.

A2.3.3 Likelihood of transverse cracking or hydraulic fracture due to differential settlement in the foundation under the core (IM11)

Refer to Section A2.2.3 which covers this mode as well as cracking and hydraulic fracture in the upper part of the embankment.

A2.3.4 Likelihood of transverse cracking or hydraulic fracture at the foundation contact due to small scale irregularities in the foundation profile under the core (IM12)

Small scale irregularities in the foundation profile under the core may comprise steps, benches or depressions in the foundation rock. Small scale irregularities include those

Table A2.20 Factors influencing the likelihood of cracking or hydraulic fracturing in the middle and lower parts of embankment dams due to small scale irregularities in the foundation profile under the core (IM12).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Persistence of the irregularity across the core	(3)	Persistent across less than 50% of the core[(b)]	Persistent 50% to 75% across the core width	Persistent 75% to 90% across the core	Persistent 90% to 100% across the core
Small scale irregularities in abutment profile	(2)	Uniform abutment profile, or irregularities treated by slope modification	Steps, benches, depressions in rock foundations less than 3% of the embankment height	Steps, benches, depressions in rock foundations 3% to 5% of the embankment height	Steps, benches, depressions in rock foundation 5% to 10% of the embankment height[(a)]
Core geometry Width (W)/ Height (H)	(1)	Wide core, $W/H > 1.5$	$0.5 < W/H < 1.5$	Narrow core, $0.25 < W/H < 0.5$	Very narrow core, $W/H < 0.25$

Notes: (a) Larger irregularities are covered in Section A2.3.2.
(b) An irregularity with less than 50% persistence across the core is assumed to have a negligible contribution to the probability of a transverse crack.

Table A2.21 Probability of a transverse crack or hydraulic fracture in the middle and lower parts of embankment dams due to small scale irregularities in the foundation profile under the core versus $\sum$ (Relative importance factor (RF)) x (Likelihood factor (LF)).

Negligible	negligible	negligible 0.0002	[0.0007]	0.002	0.01	Below POR
Negligible	negligible	negligible 0.002	[0.005]	0.02	0.1	Above POR
6	9	11	12	13	18	24 RF x LF

Notes: (1) Larger irregularities are covered in Section A2.3.2.
(2) The probability of a transverse crack across the core is negligible for persistence less than 50%.
Note: "POR" refers to the Pool of Record level + 0.3 m.

features which have heights less than 10% of the embankment height. Larger scale features are covered in Section A2.3.2.

A2.3.5 Probability of transverse cracking or hydraulic fracture – factors to account for observations and measured settlements

Settlement factors

Where there are settlement observations for the dam these can be used to modify the results of Sections A2.1 to A2.2.3, and Sections A2.3.1 to A2.3.3. The probability from the relevant table is multiplied by the factor from Table A2.22. For dams which have experienced settlements larger than the average population of that class of dam the probabilities of cracking or hydraulic fracture will be increased, and for those which have experienced settlements smaller than the average population of that class of dams the probability of cracking or hydraulic fracture will be reduced. The multiplication factor should be selected taking account of what data is available, allowing for the quantity and quality of the data and the relative importance of the observations. Table A2.22 applies to the assessment of transverse cracking in the upper part, and middle and lower part of the embankment, but lower corrections apply for the latter.

Select the likelihood column in Table A2.22 which corresponds to the maximum settlement measured anywhere in the embankment expressed as a ratio of the maximum embankment height. Then obtain the settlement multiplication factors for the upper and middle and lower parts from the corresponding bottom four rows of Table A2.22 depending on whether the dam has poorly compacted rockfill shells or not.

If there are significant differential settlements across the valley greater than expected from the mechanisms present, an additional increase in probabilities may be applied. The maximum multiplier should be less than 10 times and should not "double up" on the factors listed in Table A2.22.

Table A2.22 Settlement multiplication factors versus observed settlements.

		Influence on Likelihood			
Factor		*Less Likely*	*Neutral*	*More Likely*	*Much More Likely*
Observed maximum settlements as percentage of embankment height					
– Core settlement during construction		<1.5%	1.5% to 3%	3% to 4%	>4%
– Post construction crest settlement at 10 years after construction dams with poorly compacted shoulders		<0.5%	0.5% to 1.0%	1.0% to 1.5%	>1.5%
– Post construction crest settlement at 10 years after construction other dams		<0.25%	0.25% to 0.5%	0.5% to 1%	>1%
– Long term settlement rates (% per log time cycle in years) dams with poorly compacted shoulders		<0.15%	0.15% to 0.4%	0.4% to 0.7%	>0.7%
– Long term settlement rates (% per log time cycle in years)-other dams		<0.1%	0.1% to 0.25%	0.25% to 0.5%	>0.5%
Settlement multiplication factors for cracking or hydraulic fracture in the upper part[(a)] of the embankment based on observed maximum settlements	Dams with poorly compacted rockfill[(b)]	0.05 to 0.2	0.2 to 0.5	1.0	2 to 5
	All other dams	0.2 to 0.5	1.0	2 to 10	10 to 20
Settlement multiplication factors for cracking or hydraulic fracture in the middle and lower parts[(c)(d)] of the embankment	Dams with poorly compacted rockfill[(b)]	0.2	0.2 to 0.5	1.0	2 to 5
	All other dams	0.5	1.0	2 to 5	5 to 10

Notes: (a) Multiplication factors to be applied to Probabilities from Sections A2.3.1, A2.2.2 and A2.2.3.
(b) Includes dumped rockfill, and rockfill and other granular zones compacted by tracking with bulldozers and by small rollers in thick layers.
(c) To be applied to probabilities from Sections A2.3.1, A2.3.2 and A2.3.3.
(d) Multiplication factors assumed to be half those for cracking in the upper part.

Observation of cracking factor

Where there are observations of cracking for the dam these can be used to modify the results of Sections A2.2.1 to A2.2.5. The probability is multiplied by the factor from Table A2.23.

Evaluate the Cracking Factors (A) and (B) from Table A2.23 and then multiply the largest value of (A) or (B) to the probabilities of transverse cracking in the upper part of the dam. For dams which display cracking the probabilities will be increased. For those which do not display cracking the probabilities may remain the same, or are reduced depending on how extensive the investigations to locate cracking have been. The multiplication factor should be selected taking account of what data is available, allowing for the relative importance of the observations. This factor only applies to

Table A2.23 Cracking observation factors (applies to upper embankment only).

	Influence on Likelihood			
Factor	*Less Likely*	*Neutral*	*More Likely*	*Much More Likely*
Cracking observed in test pits to the top of or into the core	No cracking observed when large areas of the top of the core are exposed.	No test pits	Transverse cracks persistent across the top of the core and/or, extensive, open longitudinal cracking	Transverse cracks which pits show persist across the core, and extend below reservoir water level in the reservoir level partition being considered
Cracking Factor **(A)**	0.5 to 0.1 depending on the extent of exposure and how relevant the exposure is to the possible mechanism of cracking	1.0	5 to 100 depending on width(2) of cracking and whether they are in locations in which cracking might be expected	Probability of transverse crack = 1.0
Cracking in the surface of the crest, no test pits	No cracking observed, core exposed on the surface, careful inspection for cracking	No cracking observed, core covered with road pavement or other granular material	Narrow (<10 mm) transverse cracks persistent across the crest and/or, extensive, narrow longitudinal cracking	Transverse cracks which persist across the crest and/or, extensive, wide longitudinal cracking.
Cracking Factor **(B)**	0.5 to 0.2 depending on the quality of exposure and whether they are in locations in which cracking might be expected	1.0	2 to 5 depending on and whether they are in locations in which cracking might be expected	2 to 20 depending on the width(2) of cracking and whether they are in locations in which cracking might be expected

Notes: (1) Apply either Cracking Factor (A) or Cracking Factor (B), whichever gives greatest probability of cracking.
(2) The greater the crack width the more likely it represents cracking in the core.

the assessment of transverse cracking in the upper part of the embankment. Cracking in the middle and lower parts will generally not be observed.

A2.4 ESTIMATION OF THE PROBABILITY EROSION WILL INITIATE IN A CRACK OR HYDRAULIC FRACTURE IN AN EMBANKMENT (P_{IC})

A2.4.1 Overall approach

(a) **For cracking in the upper part of the dam** the method to be followed is:

- Estimate maximum likely width of cracking at the top of the core from Table A2.24.
- Estimate likely crack width at the reservoir level stage under consideration using Table A2.25 and Figure A2.7. The width required is the width at the mid level of

the flow path for the reservoir stage under consideration. In Figure A2.7 it is the average of C_{d1} and C_{d2}.

- The width C_{d1} at depth $d_1 = C_{max}((D - d_1)/D)$. That is, the crack is assumed to be uniformly tapered from the dam crest to the base of the crack.
- Estimate the likely crack width from hydraulic fracture at the reservoir level stage under consideration using Table A2.26.
- Estimate the gradient of flow through the crack using Figure A2.7. The gradient required is the average gradient at the mid level of the flow path for the reservoir stage under consideration. In Figure A2.7 it is $(d_2 - d_1)/L$.
- Assess the soil classification and whether the soils are dispersive. Dispersive soils are soils with Sherard Pinhole test D1 or D2. While reservoir water salinity will

Table A2.24 Maximum likely width of cracking at the dam crest versus $\sum$ (Relative importance factor) x (Likelihood factor) for cracking in the upper part of the dam.

	Maximum likely crack width at the dam crest in millimeters relative to (relative importance factor) x (likelihood factor)				
Crack formation mechanism	*6 to 9*	*9 to 11*	*11 to 13*	*13 to 18*	*18 to 24*
Cross valley differential settlement Table A2.1	1	20	50	75	100
Differential settlement adjacent to a spillway or other wall or cliff Table A2.3 Table A3.21	1	10	25	37	50
Cross section settlement due to poorly compacted shoulders Table A2.7	1	20	50	75	100
Differential settlements in the soil foundation Table A2.9	1	20	50	100	150
Desiccation cracking at the crest by drying Table A2.11	2	5	20	50	75
Desiccation cracking on seasonal shutdown layer Table A2.14	2	5	20	50	75

Table A2.25 Likely crack width at the depth shown versus maximum crack width at the dam crest determined from Table A2.24 for cracking in the upper part of the dam (Depths in meters).

Maximum crack width at dam crest (millimeters)	*Likely crack width at depth shown, crack width in millimeters*(1)					
	1.5 meters	*3 meters*	*4.5 meters*	*6 meters*	*7.5 meters*	*10 meters*
10	1					
25	2	1				
50	20	5	1			
75	40	20	5	2		
100	60	35	15	7	3	1
250	210	180	140	110	90	60

Note. (1) Check potential crack width resulting from hydraulic fracture and use the larger of the crack widths from this Table And Table A2.26.

affect dispersion, the salt content of the reservoir water will in most cases reduce with flood inflows and unless laboratory testing is carried out to assess the initial shear stress with the same reservoir water salts content it should be assumed soils which test dispersive in the laboratory will be so in the dam. Where there are signs of dispersive soils in field performance, e.g. severe gully erosion, sinkholes and tunneling, soils should be assumed dispersive regardless of laboratory test results.

- Based on the soil classification, dispersivity, and estimated crack width (the larger of the estimates from Table A2.25, and Table A2.26), estimate the probability of initiation of erosion in the crack and/or hydraulic fracture (P_{IC}) using Table A2.29 to Table A2.35, depending on the gradient of flow through the crack. Use approximate interpolation between values where necessary. These tables assume the relationship between erosion rate index and soil classification shown in Table A2.27.

Table A2.26 Examples of estimated maximum depths below the dam crest and widths of cracks formed by potential hydraulic fracture for cracking in the upper part of the dam.

(A) Embankment abutment

Abutment slope degrees	*Ratio of bench width to embankment height*	*Ratio of depth of zero stress to embankment height at the abutment*	*Ratio of approximate maximum depth at which hydraulic fracture may occur to embankment height at the abutment*	*Likely width of crack formed by hydraulic fracture in millimeters*
15	0.67	<0.01	0.05	2
25	No bench	0.02	0.05	2
45	No bench	0.12	0.3	5
45	0.2	0.12	0.3	5
45	0.4	0.09	0.25	5
45	1.0	0.10	0.25	5
60	No bench	0.35	0.5	10

(B) Adjacent to walls and cliffs in the upper part of the embankment

$\sum$ (Relative importance factor (RF)) x (Likelihood factor (LF)) from Table A5.3	*Likely width of crack formed by hydraulic fracture in millimeters*
14 to 19	5
>19	10

(C) Cross valley arching in the upper part of the embankment

$\sum$ (Relative importance factor (RF)) x (Likelihood factor (LF)) from Table A2.5	*Likely width of crack formed by hydraulic fracture in millimeters*
10 to 13	2
14 to 19	5
>19	10

(D) Over low stress zones over irregularities in the foundation

Assume that hydraulic fracture may persist from the crest to the foundation level, and the width is 5 mm.

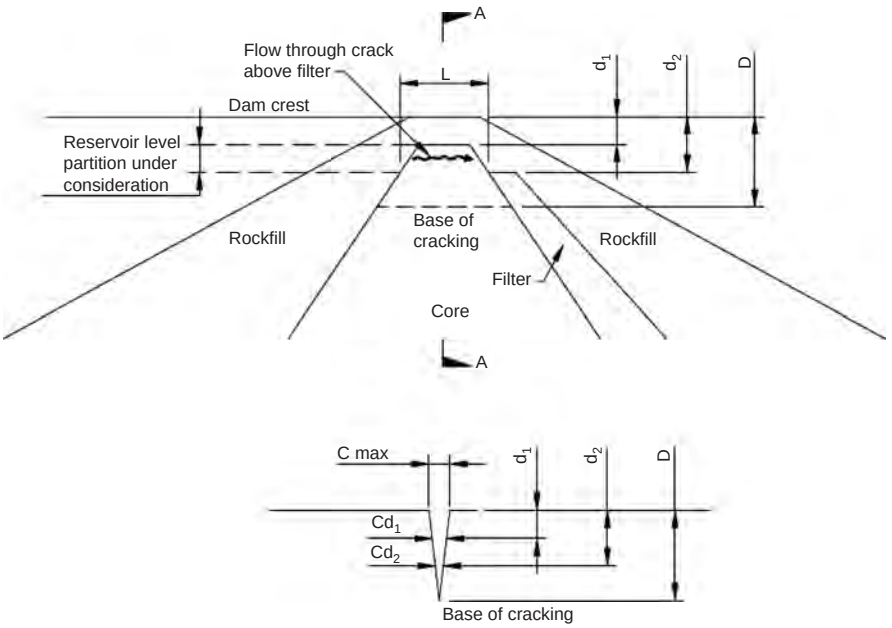


Figure A2.7 Example of the estimation of crack width and flow gradient in the crack.

Table A2.27 Representative erosion rate index (I_{HET}) versus soil classification for non dispersive soils based on Wan and Fell (2002, 2004).

	Representative Erosion Rate Index (I_{HET})		
Soil Classification	*Likely Minimum*	*Best Estimate*	*Likely Maximum*
SM with <30% fines	1	<2	2.5
SM with >30% fines	<2	2 to 3	3.5
SC with <30% fines	<2	2 to 3	3.5
SC with >40% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit <65%	3	4	5
CH with Liquid Limit >65%	4	5	6

Note.
(1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.
(3) The Representative Erosion Rate index is for soils compacted to 95% standard (Proctor) maximum dry density at optimum moisture content.
(4) See Supporting Information Report for information regarding the Representative Erosion Rate index for soils which are significantly drier than optimum moisture content or saturated.

These tables only apply to soil compacted to 95% to 98% of Standard Proctor maximum dry density at a moisture content between −1% to +2% of optimum moisture content. For saturated soils, soils significantly dry of optimum moisture content, and poorly compacted soils, Hole Erosion Tests should be carried out to determine the initial shear stress, and the method detailed in (c) below followed.

- If there are Hole Erosion Tests available use the method detailed in (c) below.

(b) For cracking in the middle and lower parts of the dam
The method to be followed is:

- Estimate maximum likely width of cracking in the middle or lower part of the dam from Table A2.28. This includes hydraulic fracture.
- Assess the soil classification and whether the soils are dispersive. Dispersive soils are soils with Sherard Pinhole test D1 or D2. While reservoir water salinity will affect dispersion, the salt content of the reservoir water will in most cases reduce with flood inflows and unless laboratory testing is carried out to assess the initial shear stress with the same reservoir water salts content it should be assumed soils which test dispersive in the laboratory will be so in the dam. Where there are signs of dispersive soils in field performance, e.g. severe gully erosion, sinkholes and tunneling, soils should be assumed dispersive regardless of laboratory test results.
- Based on the soil type and estimated crack width, estimate the probability of initiation of erosion in the crack and/or hydraulic fracture (P_{IC}) using Table A2.29 to Table A2.35, depending on the gradient across the crack. If there are hole erosion tests available assess which classification should apply to best reflect this

Table A2.28 Maximum likely width of cracking in the dam versus $\sum$(Relative importance factor) x (Likelihood factor) for cracking in the middle and lower parts of the dam.

	Maximum likely crack width in millimeters relative to (relative importance factor) x (likelihood factor)				
Crack formation mechanism	*6 to 9*	*9 to 11*	*11 to 13*	*13 to 18*	*18 to 24*
Cross valley differential settlement Table A2.16	0	0	1 to 2	2 to 10	10 to 20
Differential settlement causing arching of the core onto the shoulders Table A2.18	0	0	0 to 1	1 to 2	2 to 10
Differential settlement over small scale irregularities in the foundation Table A2.20	0	0	0 to 1	1 to 2	2 to 10
Differential settlements due to settlements in the foundation Table A2.9	0	0	0 to 2	2 to 10	10 to 20
Differential settlement causing arching of the core in the cut off trench Table A9.3 of Piping Toolbox	0	0	0 to 1	1 to 2	2 to 10

value when using Table A2.29 to Table A2.35. Use approximate interpolation between values where necessary. These tables only apply to soil compacted to 95% to 98% of Standard Proctor maximum dry density at a moisture content between −1% to +2% of optimum moisture content. For saturated soils, soils significantly dry of optimum moisture content, and poorly compacted soils, Hole Erosion Tests should be carried out to determine the initial shear stress, and the method detailed in (c) below followed.

Table A2.29 Estimation of probability of initiation in a crack for ML or SM with <30% fines soil types.

	Probability of initiation of erosion for different seepage gradients					
Estimated likely crack width in core for Reservoir stage being considered (mm)	*Average Hydraulic Gradient*					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.05	0.2	0.6	0.95	1.0	1.0
2	0.1	0.6	0.9	1.0	1.0	1.0
5	0.6	1.0	1.0	1.0	1.0	1.0
10	0.9	1.0	1.0	1.0	1.0	1.0
25	1.0	1.0	1.0	1.0	1.0	1.0
50	1.0	1.0	1.0	1.0	1.0	1.0
75	1.0	1.0	1.0	1.0	1.0	1.0
100	1.0	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

Table A2.30 Estimation of probability of initiation in a crack for SC with <40% fines, or SM with >30% fines soil types.

	Probability of initiation of erosion for different seepage gradients					
Estimated likely crack width in core for Reservoir stage being considered (mm)	*Average Hydraulic Gradient*					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.02	0.2	0.6	0.9	0.95	1.0
2	0.1	0.6	0.9	0.95	1.0	1.0
5	0.6	0.95	0.99	1.0	1.0	1.0
10	0.9	1.0	1.0	1.0	1.0	1.0
25	0.95	1.0	1.0	1.0	1.0	1.0
50	1.0	1.0	1.0	1.0	1.0	1.0
75	1.0	1.0	1.0	1.0	1.0	1.0
100	1.0	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

(c) **Procedure to be followed where Hole Erosion Test data is available**
In cases where Hole Erosion tests are available for the dam core soil, the following procedure should be followed:

- Estimate the crack width and hydraulic flow gradient as detailed in (a) or (b), whichever is applicable.
- EITHER Calculate the hydraulic shear stress in the crack for the reservoir stage under consideration using Table A2.36.
- AND Compare this hydraulic shear stress to the initial shear stress of the soil at the compaction and moisture conditions it exists in the core. Based on this comparison estimate the probability of initiation of erosion. In doing this calculation take

Table A2.31 Estimation of probability of initiation in a crack for SC with >40% fines, or CL-ML soil types.

	Probability of initiation of erosion for different seepage gradients					
Estimated likely crack width in core for Reservoir stage being considered (mm)	*Average Hydraulic Gradient*					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.02	0.1	0.4	0.8	0.9	0.95
2	0.1	0.5	0.7	0.9	0.95	1.0
5	0.4	0.8	0.9	0.95	1.0	1.0
10	0.7	0.9	0.95	1.0	1.0	1.0
25	0.9	0.95	1.0	1.0	1.0	1.0
50	0.95	1.0	1.0	1.0	1.0	1.0
75	1.0	1.0	1.0	1.0	1.0	1.0
100	1.0	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

Table A2.32 Estimation of probability of initiation in a crack for CL or MH soil types.

	Probability of initiation of erosion for different seepage gradients					
Estimated likely crack width in core for Reservoir stage being considered (mm)	*Average Hydraulic Gradient*					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.01	0.03	0.1	0.2	0.3	0.7
2	0.02	0.1	0.2	0.5	0.6	0.9
5	0.1	0.3	0.5	0.7	0.9	1.0
10	0.2	0.5	0.7	0.95	1.0	1.0
25	0.4	0.7	0.95	1.0	1.0	1.0
50	0.7	1.0	1.0	1.0	1.0	1.0
75	0.9	1.0	1.0	1.0	1.0	1.0
100	0.95	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

account of the uncertainty in the crack width, and initial shear stress detailed in Section S5.4.2.4 of the Piping Toolbox Supporting Document.

- OR/AND Use Table A2.37 to determine which of Tables A2.29 to A2.35 best fits the initial shear stress of the soil tested in the HET and use that table to estimate the probability of initiation of erosion.

A2.4.2 Details of the method

The crack width values given in Table A2.24 represent maximum values from case studies but the method assumes a range of crack widths varying from 25% to 100% of the maximum crack widths shown in the table.

Table A2.33 Estimation of probability of initiation in a crack for CL-CH or CH with LL<65% soil types.

Estimated likely crack width in core for Reservoir stage being considered (mm)	*Probability of initiation of erosion for different seepage gradients*					
	Average Hydraulic Gradient					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.005	0.02	0.05	0.1	0.2	0.6
2	0.01	0.05	0.1	0.3	0.5	0.9
5	0.05	0.2	0.3	0.6	0.8	1.0
10	0.1	0.3	0.6	0.9	0.95	1.0
25	0.3	0.6	0.9	1.0	1.0	1.0
50	0.6	0.95	1.0	1.0	1.0	1.0
75	0.8	1.0	1.0	1.0	1.0	1.0
100	0.9	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

Table A2.34 Estimation of probability of initiation in a crack for CH with LL > 65% soil types.

Estimated likely crack width in core for Reservoir stage being considered (mm)	*Probability of initiation of erosion for different seepage gradients*					
	Average Hydraulic Gradient					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.001	0.002	0.005	0.01	0.05	0.1
2	0.002	0.005	0.01	0.02	0.1	0.4
5	0.005	0.01	0.05	0.1	0.3	0.95
10	0.01	0.04	0.1	0.4	0.6	1.0
25	0.02	0.1	0.4	0.8	0.95	1.0
50	0.1	0.5	0.95	1.0	1.0	1.0
75	0.3	0.8	1.0	1.0	1.0	1.0
100	0.4	0.95	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

Table A2.35 Estimation of probability of initiation in a crack for dispersive soils (CL, CH, CL-CH).

Estimated likely crack width in core for Reservoir stage being considered (mm)	*Probability of initiation of erosion for different seepage gradients*					
	Average Hydraulic Gradient					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.02	0.1	0.3	0.6	0.8	1.0
2	0.05	0.3	0.6	0.9	1.0	1.0
5	0.3	0.7	1.0	1.0	1.0	1.0
10	0.5	1.0	1.0	1.0	1.0	1.0
25	1.0	1.0	1.0	1.0	1.0	1.0
50	1.0	1.0	1.0	1.0	1.0	1.0
75	1.0	1.0	1.0	1.0	1.0	1.0
100	1.0	1.0	1.0	1.0	1.0	1.0

Note. (1) The gradient is the average hydraulic gradient from the upstream to the downstream of the core at the level of the assumed crack under the reservoir level under consideration. No allowance is made for seepage head losses in the zones upstream or downstream of the core.

Table A2.36 Estimated hydraulic shear stress (N/m^2) from water flowing in an open crack, versus crack width and flow gradient.

Crack Width Millimeters	*Flow Gradient in Crack*					
	0.1	*0.25*	*0.5*	*1.0*	*2.0*	*5.0*
1	0.5	1.25	2.5	5	10	25
2	1	2.5	5	10	20	50
5	2.5	6	12	25	50	125
10	5	12	25	50	100	250
20	10	25	50	100	200	500
50	25	60	125	250	500	1250
100	50	125	250	500	1000	2500

Table A2.37 Initial Shear Stress assumed for Tables A2.29 to A2.35.

Table Number	*Soil Types*	*Initial Shear Stress Assumed for Assessing Probabilities of Initiation of Erosion*
A2.29	ML and SM with <30% fines	2 Pa
A2.30	SC with, <40% fines, SM with >30% fines	2 Pa
A2.31	SC with >40% fines, and CL-ML	4 Pa
A2.32	CL and MH	5 Pa
A2.33	CL-CH and CH with LL <65%	25 Pa
A5.34	CH with LL >65%	60 Pa
A5.35	Dispersive soils	2 Pa

A2.5 ESTIMATION OF THE PROBABILITY OF TRANSVERSE CRACKS IN THE EMBANKMENT CAUSED BY EARTHQUAKE (IM13)

- **First** – Determine the earthquake hazard for the site (refer to Section 12.3 of Chapter 12 for details).

- **Second** – Estimate the likely damage class which the embankment may experience as a result of the from the earthquake and peak ground acceleration on bedrock at the dam site using Figure A2.8 for earthfill dams or Figure A2.9 for earth and rockfill dams. Do this for each earthquake load partition. The earthquake load partition ranges should be selected such that they coincide with the damage class contours shown in Figure A2.8 for earthfill dams or Figure A2.9 for earthfill and rockfill dams.

Table A2.38 Damage classification system (Pells and Fell, 2002, 2003).

Damage Class		*Maximum Longitudinal Crack Width*(1) *(mm)*	*Maximum Relative Crest Settlement*(2) *(%)*
Number	*Description*		
0	No or Slight	<10 mm	<0.03
1	Minor	10–30	0.03–0.2
2	Moderate	30–80	0.2–0.5
3	Major	80–150	0.5–1.5
4	Severe	150–500	1.5–5
5	Collapse	>500	>5

(1) Maximum crack width is taken as the maximum width, in millimeters, of any longitudinal cracking that occurs.
(2) Maximum relative crest settlement is expressed as a percentage of the structural dam height.

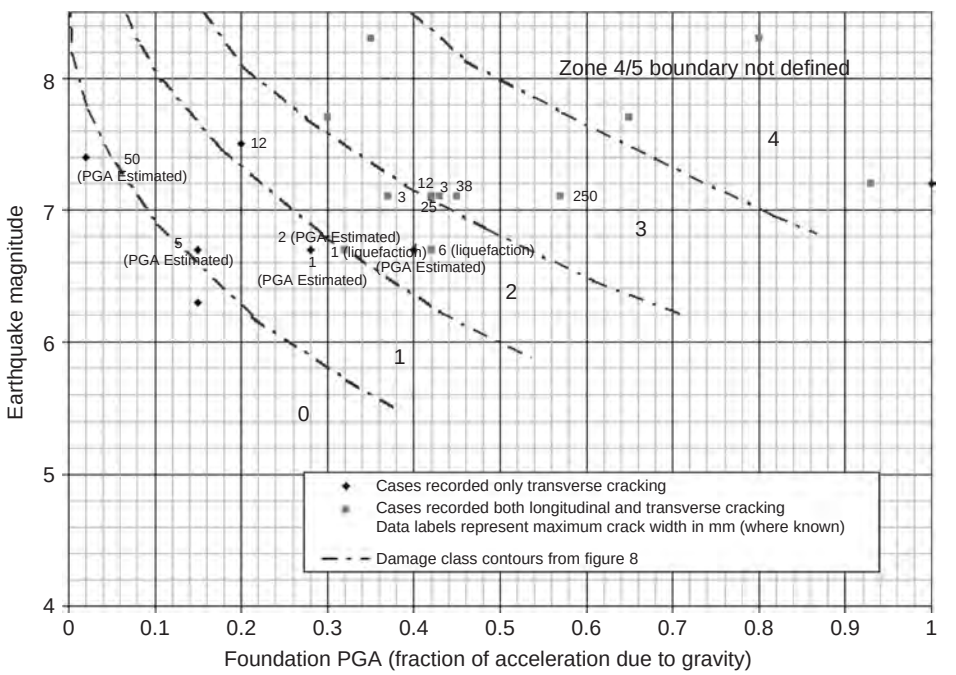


Figure A2.8 Incidence of transverse cracking versus seismic intensity and damage class contours for earthfill dams (Pells and Fell 2002, 2003).

- **Third** – Estimate the probability of transverse cracking and likely maximum crack width from the damage class and Table A2.39.
- **Fourth** – Estimate the likely crack depth and assess the probability erosion will initiate using the method described in Section A2.4.

This procedure applies to situations where liquefaction does not occur in the dam or its foundations. If flow liquefaction occurs assume the damage is class 4. For cases where liquefaction occurs but it is not flow liquefaction, assume damage class 3.

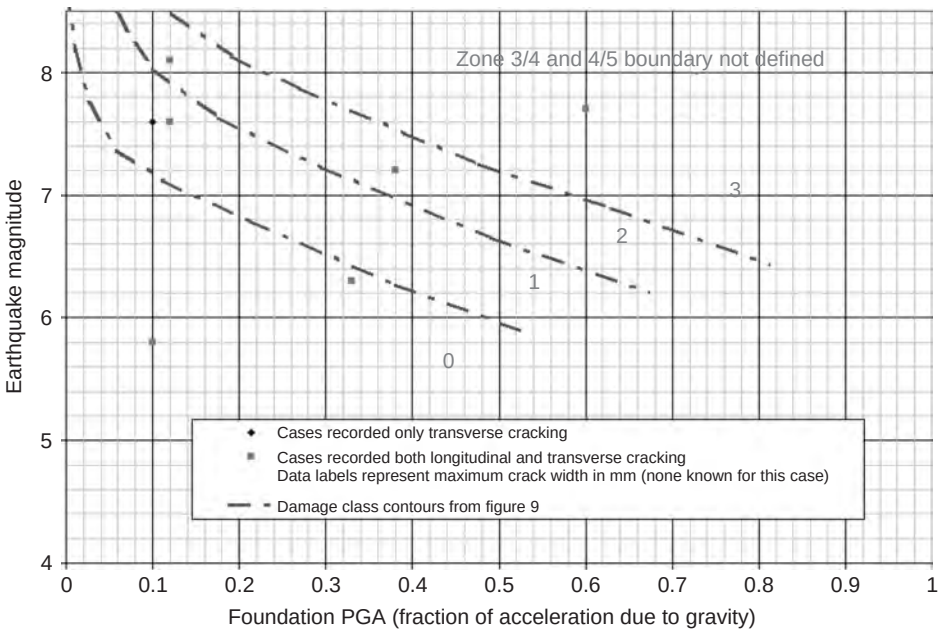


Figure A2.9 Incidence of transverse cracking versus seismic intensity and damage class contours for earthfill and rockfill dams (Pells and Fell 2002, 2003).

Table A2.39 Estimation of the probability of transverse cracking from the damage class.

	For cases where cross valley or cross section cracking assessment is in lower three "boxes" i.e. RFxLF ≤ 12 in Tables A2.2, A2.4 and A2.10		*For cases where cross valley or cross section cracking assessment is in upper two "boxes" i.e. RF x LF > 13 in Tables A2.2, A2.4 and A2.10*	
Damage class	*Probability of transverse cracking*	*Maximum likely crack width mm*	*Probability of transverse cracking*	*Maximum likely crack width at crest mm*
0	0.001	5	0.01	20
1	0.01	20	0.05	50
2	0.05	50	0.10	75
3	0.2	100	0.25	125
4	0.5	150	0.6	175

A3 Probability of Initiation of Erosion in Poorly Compacted Zones in the Embankment

A3.1 OVERALL APPROACH

a) Estimate the probability of a continuous poorly compacted or high permeability zone (P_P) for each of the mechanisms which can lead to a poorly compacted or high permeability zone. These are:
   - Poor compaction or high permeability layer during construction within the core (Section A3.2.1);
   - Poor compaction or high permeability layer on the foundation contact (Section A2.21);
   - Poor compaction or high permeability layer due to freezing (Section A3.2.2);
   - Poor compaction or high permeability layer around a conduit or other structure penetrating the core (Sections A3.3 and A3.4).

   Adjust this estimate to account for observations (Section A3.5).

b) Assess the erosion mechanism(s) which will apply. This will be one or more of the following:
   - Backward erosion
   - Suffusion
   - Erosion in a crack or flaw resulting from the poor compaction.

   Backward erosion and suffusion will apply to cohesionless soils and as discussed in Section A3.6.1, to soils with a plasticity index ≤ 7. For cohesive soils, erosion will occur in cracks or continuous open flow paths formed by collapse of the soil on saturation, or between aggregated particles of the soil, and can be considered as equivalent to erosion in a crack. Even low plasticity soils may form a crack so soils with a plasticity index between zero and 7 should be considered for both erosion in a crack and backward erosion and suffusion and the highest probability of initiation carried forward in the analysis.

c) Assess the probability of erosion in the poorly compacted or high permeability zone (P_{IP}) for the mechanism using the relevant method described in Section A3.6.

d) Estimate the probability of initiation of erosion (P_I) = (P_P) x (P_{IP}) for each poorly compacted or high permeability zone. For many dams one or more of the mechanisms will not be present.

A3.2 ESTIMATION OF THE PROBABILITY OF A CONTINUOUS POORLY COMPACTED OR HIGH PERMEABILITY ZONE IN THE EMBANKMENT OR ON THE CORE-FOUNDATION CONTACT

A3.2.1 Poorly compacted or high permeability zone within the core (IM14)

The scenarios which may lead to poorly compacted or high permeability zones in the core of an embankment include:

- Poorly compacted layers in the core.
- A segregated layer in the core due to the presence of coarser particles and poor construction practices.

Table A3.1 Factors influencing the likelihood of poorly compacted or high permeability zones in the embankment-cohesive soils (IM14).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)* *Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
METHOD BASED ON COMPACTION EQUIPMENT, LAYER THICKNESS AND MOISTURE CONTENT[a]					
Compaction equipment	(3)[a]	As for "neutral" but with good documentation and records[2]	Soil compacted by suitable rollers in suitable layer thicknesses	Soil placed and compacted by bulldozer, no compaction by rollers, or rolled in thick layers beyond the capability of the roller	Soil placed with, no formal compaction (e.g. by horse and cart in old dams, or by pushing into place by excavator or bulldozer or in very thick layers)
and Layer thickness			Layer thickness 150 mm to 250 mm after compaction	Layer thickness at or beyond the limit of compaction equipment (e.g. >300 mm to 450 mm after compaction)	No control on layer thickness, often >450 mm to 600 mm loose
and moisture content			Around optimum moisture content[1]	Dry of optimum moisture content[1]	Well dry of optimum moisture content.[1]
METHOD BASED ON COMPACTION EQUIPMENT, LAYER THICKNESS AND MOISTURE CONTENT[a]					
AND/OR[a] Measured or estimated compaction density ratio and moisture content (COHESIVE SOILS)	(3)[a]	All very well compacted to e.g. ≥98% standard dry density ratio, moisture content 2% dry of optimum to 1% wet of standard OWC[b]	Well compacted to e.g. 95–98% standard dry density ratio, moisture content 2% dry of optimum to 1% wet of standard OWC	Layers poorly compacted, dry of standard optimum moisture content e.g. <93% standard dry density ratio, 2% to 3% dry of standard OWC	Layers very poorly compacted, dry of standard optimum moisture content e.g. <90% standard dry density ratio, 3% dry of standard OWC
FACTORS APPLYING TO BOTH ALTERNATIVES					
Borrow area variability, Site supervision	(2)	Uniform soils in the borrow areas, Good site supervision documented with laboratory tests,	Uniform or minor variability in the borrow areas, Good site supervision	Variable soils in the borrow areas Moderate site supervision	Very variable soils in borrow areas including gravely soils; Poor site supervision,
Core geometry Width (W)/Height (H)	(1)	Wide core, W/H>1.5	0.5<W/H<1.5	Narrow core, 0.25<W/H<0.5	Very narrow core, W/H<0.25

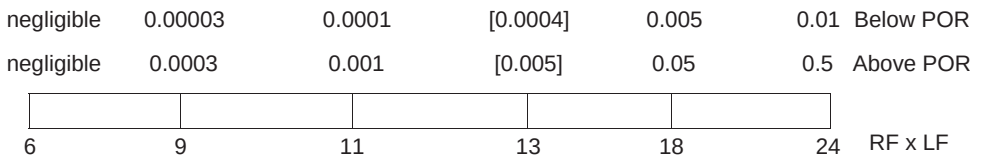
Notes: (a) Make an assessment based on a combination of available data.
(b) If soils are well compacted this mechanism will not apply.

Table A3.2 Factors influencing the likelihood of poorly compacted or high permeability zones in the embankment-non cohesive soils (IM14).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)* *Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
EITHER[a] Compaction equipment	(3)[a]	As for "neutral" but with good documentation and records[b]	Soil compacted by suitable rollers in suitable layer thicknesses	Soil placed and compacted by bulldozer, no compaction by rollers, or rolled in thick layers beyond the capability of the roller	Soil placed with, no formal compaction (e.g. by horse and cart in old dams, or by pushing into place by excavator or bulldozer or in very thick layers
and Layer thickness			Layer thickness 200 mm–300 mm after compaction	Layer thickness at or beyond the limit of compaction equipment (e.g. >300 mm to 450 mm after compaction)	No control on layer thickness, often >600 mm to 900 mm loose
and moisture content			Around optimum moisture content[a]	Dry of optimum moisture content[a]	Well dry of optimum moisture content[a]
AND/OR[a] Measured or estimated compaction density ratio and moisture content (NON-COHESIVE SOILS)	(3)[a]	All very well compacted e.g. very dense, >85% relative density with good documentation and records[b]: SPT $(N_1)_{60} > 42$	All well compacted e.g. dense, 66% to 85% relative density: SPT $(N_1)_{60}$ 26 to 42	Layers moderately compacted, e.g. medium dense, 36% to 65% relative density: SPT $(N_1)_{60}$ 9 to 25	Layers very poorly compacted, e.g. very loose to loose, <35% relative density, SPT $(N_1)_{60} < 8$
Borrow area variability, Site supervision	(2)	Uniform soils in the borrow areas, Good site supervision documented with laboratory tests,	Uniform or minor variability in the borrow areas, Good site supervision	Variable soils in the borrow areas Moderate site supervision	Very variable soils in borrow areas including gravely soils; Poor site supervision,
Core geometry Width (W)/Height (H)	(1)	Wide core, W/H>1.5	0.5<W/H<1.5	Narrow core, 0.25<W/H<0.5	Very narrow core, W/H<0.25

Notes: (a) Make an assessment based on a combination of available data.
(b) If soils are well compacted this mechanism will not apply.

Table A3.3 Probability of a poorly compacted or high permeability layer in the embankment versus $\sum$ (Relative importance factor (RF)) x (Likelihood factor (LF)) (applies to upper, middle and lower parts of the embankment).



Note: "POR" refers to the Pool of Record level + 0.3 m.

- Coarser soil layers in the core due to variability in particle size and soil type in the borrow areas. This can result in a cohesionless layer within a cohesive core, or a coarser cohesionless layer within a finer cohesionless soil.

The effect of these factors on the likelihood of a poorly compacted or high permeability zone is different for cohesive and cohesionless soils so they are treated separately. For these purposes a cohesionless soil is non plastic.

A3.2.2 Poorly Compacted or High permeability layer on the core-foundation contact (IM15)

Table A3.4 Factors influencing the likelihood of poorly compacted or high permeability zones on the core-foundation/abutment contact (IM15).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
EITHER Foundation preparation below the core, rock foundations,	(3)	Uniform rock surface or surface treated with shotcrete, or concrete to correct slope irregularities[(c)]	Regular rock surface, or rock surface treated with shotcrete or concrete to correct slope irregularities[(b)]	Irregular rock surface, with minimal slope correction or treatment or irregular or benched soil with no compaction	Very irregular rock surface, overhangs with no slope correction, shotcrete or concrete treatment
OR Foundation preparation below the core, soil foundations	(3)	Uniform well compacted soil foundation[(c)]	Compacted soil foundation[(b)]	Irregular or benched soil with no compaction	Poor stripping of soil foundation leading to poor compaction of first lift

(*Continued*)

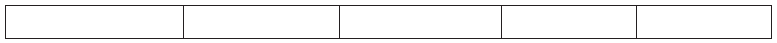
Table A3.4 Continued.

Factor	Relative Importance Factor (RF)	Likelihood Factor (LF)			
		Less Likely (1)	Neutral (2)	More Likely (3)	Much More Likely (4)
Compaction methods for contact zone	(2)	As for "neutral" but with good documentation and records[(d)]	Soil compacted using special compaction methods (rubber tires, use more plastic materials, compaction wet of OWC)	Soil placed and compacted by bulldozer, no compaction by rollers Layer thickness at the limit of compaction equipment (e.g. >300 mm or 450 mm after compaction)[(a)]	Poor compaction methods used, soil poorly compacted or allowing segregation against foundation surface. Thick layer thickness, often >450 mm or 600 mm loose
Continuity of features, Core geometry Width (W)/ Height (H)	(1)	Wide core, W/H > 1.5 Core wall cutoff or grout cap cutoff present	0.5<W/H<1.5	Narrow core, 0.25<W/H<0.5	Very narrow core, W/H<0.25

Notes: (a) Smaller thicknesses are for cohesive soil, larger thicknesses for cohesionless soil.
(b) For most cases this mechanism may be judged as of negligible likelihood and assigned zero probability.
(c) For this foundation preparation the probability of this mechanism is negligible
(d) Even situations where soil is well compacted can soften or loosen if the contact is irregular.
(e) For homogeneous earthfill dams, assess foundation preparation and compaction methods for the central portion of the section.

Table A3.5 Probability of a poorly compacted or high permeability layer on the core foundation contact versus $\sum$ (Relative importance factor (RF)) x (Likelihood factor (LF)).

Negligible	0.0001	0.0002	[0.0004]	0.005	0.01	Below POR
Negligible	0.0005	0.002	[0.005]	0.05	0.1	Above POR



6	9	11	13	18	24	RF x LF

Notes: (1) This and some other combinations which involve contradictory compaction scenarios are quite unlikely to occur
(2) If soils are well compacted they this mechanism will not apply
Note: "POR" refers to the Pool of Record level + 0.3 m.

A3.2.3 Poorly compacted or high permeability layer in the embankment due to freezing (IM16, IM17)

A3.2.3.1 *At the crest of the embankment (IM16)*

Freezing conditions can result in frost heave and formation of ice lenses in the crest of dams. When the ice thaws, loosened and/or cracked soil may be present in which internal erosion may initiate if the reservoir rises sufficiently high. This section describes

how to assess the probability of the presence of such features and the depth to which they may exist. The procedure is:

FIRST Consider the maximum likely depth of freezing for the soil in the core of the dam, and the climate using Table A3.8. If the reservoir stage being considered is below the likely depth of freezing, the probability of a crack or poorly compacted zone due to freezing can be assumed = zero.

SECOND For cases where the reservoir stage is above the base of potential freezing, follow the procedure below to estimate the probability of frost action affecting the dam and the depth of the frost action.

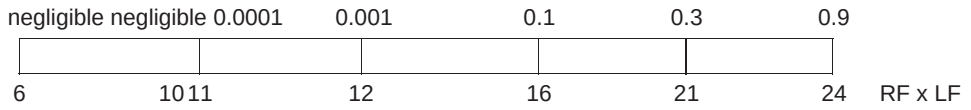
The method for estimating the maximum width of flaw which may result from the frost action is detailed in Section A3.6.6.

Table A3.6 Factors influencing the likelihood of cracking and poorly compacted zones in the upper part of embankment dams due to freezing (IM16).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)* *Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Climate	(3)	Other climates where temperatures do not fall below freezing point except possibly overnight or for a day or two.[a]	Temperate climate where temperatures may remain below freezing point for up to 1 month	Sub arctic or alpine climates where temperatures remain below freezing point for 1 to 3 months	Sub arctic or alpine climates where temperatures remain below freezing point for 3 months or more
Classification of core material	(2)	Clean gravel (GP,GW) and sand (SP,SW), less than 3% finer than 0.02 mm	Gravely (GP,GW) and sandy (SP,SW), soils with between 3% and 6% finer than 0.02 mm High plasticity clays (CH)	Silty gravely (GM,GW-GM, GP-GM) and silty sandy soils (SM, SW-SM, SP-SM) with 6% to 15% finer than 0.02 mm Clayey sands and gravels (SC, GC) and clays with plasticity index <12	Silts (ML, MH), silty sands (SM) with >15% finer than 0.02 mm, And clayey silts (ML-CL)
Crest zoning-surface layer over core	(1)	Greater than 2 meters of rockfill over the core	Gravely material or rockfill 1 m to 2 m thick over the core	Gravel material or rockfill 0.45 m to 1 m thick over the core	No surface layer with dam core extending to crest level or thin (less than 75 mm) road pavement or gravely material

Note: (a) If climatic conditions are as described here this mode does not apply.

Table A3.7 Probability of cracking or poorly compacted zones in the crest due to freezing versus ∑(Relative importance factor (RF)) x (Likelihood factor (LF)).



Note: "POR" refers to the Pool of Record level + 0.3 m.

Table A3.8 Maximum likely depth of freezing induced flaws based on climate.

Climate	*Maximum likely depth (meters)*
Sub arctic or alpine climates where temperatures remain below freezing point for 3 months or more	1.8
Sub arctic or alpine climates where temperatures remain below freezing point for 1 to 3 months	1.3
Temperate climate where temperatures may remain below	0.6
Other climates where temperatures do not fall below freezing point except possibly overnight or for a day or two.	0

A3.2.3.2 *On seasonal shutdown layers and on the surface of staged embankments (IM17)*

FIRST This mechanism only applies where there has been a seasonal shutdown during construction, or the embankment has been staged.

SECOND Where the mechanism applies estimate the probability of a freezing layer using Tables A3.9 and A3.10, and multiply this probability by the assessed

Table A3.9 Factors influencing the likelihood of high permeability layer on seasonal shutdown layers during construction and staged construction surfaces due to freezing (IM17).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Construction practices regarding clean-up of frozen layers after construction shutdowns or the surface of the earlier stage of the dam	(4)	Very good control and clean-up practices. Frozen layers removed from embankment and replaced with new soil or adequately reworked to specified moisture content. If this condition is present, probability=zero	Good control and practices, surfaces scarified, moisture adjusted to specified range, surface re-compacted.	Moderate control. Attempts to scarify frozen layers, but depth of scarifying insufficient or difficulties with moisture control on re-compacting the soil	Poor control. No attempt to scarify or remove frozen layers, poor moisture control on re-compacting the soil

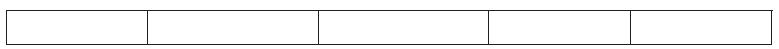
(*Continued*)

Table A3.9 Continued.

Factor	Relative Importance Factor (RF)	Likelihood Factor (LF) Less Likely (1)	Neutral (2)	More Likely (3)	Much More Likely (4)
Climate	(2)	Other climates where temperatures do not fall below freezing point except possibly overnight or for a day or two.	Temperate climate where temperatures may remain below freezing point for up to 1 month	Sub arctic or alpine climates where temperatures remain below freezing point for 1 to 3 months	Sub arctic or alpine climates where temperatures remain below freezing point for 3 months or more
Classification of core material	(1)	Clean gravel (GP,GW) and sand (SP, SW), less than 3% finer than 0.02 mm	Gravely (GP, GW) and sandy (SP,SW) soils with between 3% and 6% finer than 0.02 mm High plasticity clays (CH)	Silty gravely (GM, GW-GM, GP-GM) and silty sandy soils (SM, SW-SM, SP-SM) with 6% to 15% finer than 0.02 mm Clayey sands and gravels (SC, GC) and clays with plasticity index <12	Silts (ML, MH), silty sands (SM) with $>15\%$ finer than 0.02 mm, And clayey silts (ML-CL)

Table A3.10 Probability of a high permeability layer on seasonal shutdown layers during construction and staged construction surfaces due to freezing versus $\sum$ (Relative importance factor (RF)) x (Likelihood factor(LF)).

negligible	negligible	negligible 0.0001	0.001	0.01	0.1	Below POR
negligible	negligible	negligible 0.001	0.01	0.1	0.9	Above POR



7 10 12 13 18 22 28 RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

likelihood the layer will be continuous across the core. This likelihood is to be assessed by the risk analysis team based on the information available and their judgment.

A3.3 PROBABILITY OF A POORLY COMPACTED OR HIGH PERMEABILITY ZONE AROUND A CONDUIT OR FEATURES ALLOWING EROSION INTO THE CONDUIT

A3.3.1 Poorly compacted or high permeability zone around a conduit through the embankment (IM18)

Assess the probability of a poorly compacted or high permeability zone around the conduit using Table A3.11 and Table A3.12.

Table A3.11 Factors influencing the likelihood of poorly compacted or high permeability zones along outside of a conduit (IM18).

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Conduit type and surround	(3)	Concrete encased round pipe, concrete precast or cast in situ, sloping sides.	Concrete encased round pipe, concrete precast or cast in situ, vertical sides. Flowable fill (CLSM)	Masonry, brick,	Any round pipe (including corrugated metal pipe), not concrete encased
Cut off collars	(2)	No cut off collars	Well detailed cut off collars, USBR design	Poorly detailed cut off collars, widely spaced	Poorly detailed cut off collars, close spacing
Compaction of earthfill around the conduit	(2)	Compaction by rollers to >98% standard maximum dry density at OWC−1% to OWC+2%	Compaction by hand and mechanical equipment to >95% standard maximum dry density at OWC−1% to OWC+2%	Compaction by hand equipment, thick layers, dry of optimum moisture content	No formal compaction, or poor compaction practices used adjacent to conduits (e.g. thick layers inappropriate for equipment)
Conduit trench details	(2)	Trench totally in non-erodible rock, backfilled to the surface with concrete [(a)]	Wide, slopes flatter than 1H:1V, base width not less than conduit width plus 2 meters either side. No desiccation of sides of trench	Medium width, depth, and slope; and/or sides of trench desiccated and cracked	Narrow, deep, near vertical sides in soil or rock, backfilled with soil; and/or sides of trench highly desiccated and cracked

Note: (a) If the conduit is totally embedded in a trench totally in non-erodible rock, backfilled to the surface with concrete it should not be considered as a conduit through the embankment (probability = zero).

Table A3.12 Probability of a poorly compacted or high permeability zone associated with a conduit versus ∑(Relative importance factor (RF)) x (Likelihood factor (LF)).

0.0001	0.0002	0.0005	[0.0009]	0.005	0.02	Below POR
0.0003	0.0006	0.0015	[0.003]	0.02	0.1	> 20% Above POR
0.001	0.002	0.005	[0.01]	0.05	0.5	First fill (untested)
9	12	16	20	26	36	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.



Figure A3.1 Example of poor detailing of seepage collars around a conduit (from FEMA 2005).

A3.3.2 Features allowing erosion into a conduit (IM19)

Assess the probability of a feature being present in the conduit which would allow erosion of the surrounding soil into the conduit.

a) If the internal condition of the conduit is regularly inspected and there is good documentation of the inspections, then estimate the probability of erosion into a conduit using Table A3.13.
b) If the internal condition of the conduit is not known, then use Table A3.14.

It should be noted that this mechanism can apply for erosion into the conduit from the foundation and from the embankment.

Erosion into a conduit may be followed by development of erosion along the conduit in the progression stage. This should be considered when assessing the likelihood of breach. The likelihood of development of piping along the conduit given erosion initiates into the conduit should be estimated from Table A3.16, with the ∑(Relative importance factor (RF))×(Likelihood factor (LF)) taken from Table A3.11. The two

potential failure paths should be considered in progression, detection and intervention, and breach.

Table A3.13 Factors influencing the likelihood of initiation of erosion into a non-pressurized conduit when internal condition is known.

Observed Condition	*Probability of initiation for Erosion into the Conduit*[1]
Careful inspection showing no evidence of open joints or cracks	Negligible
Careful inspection showing no evidence of open joints, but hairline cracks in concrete are present	0.001 to 0.005
Open joints or cracks present, no evidence of seepage	0.05 to 0.3
Corroded corrugated metal, or cast iron or steel with advanced stages of corrosion[2]	0.1 to 0.9
Open joints or cracks present, evidence of seepage through joint/crack	0.3 to 0.9
Open joints or cracks present, evidence of erosion of soil into the conduit	1.0

Notes. (1) Select best estimate taking account of the width of joint openings, the type of joints, the width of cracks, and the quantity of seepage through the crack or joint.
(2) Select best estimate accounting for the extent of corrosion, and whether it penetrates through the conduit. Consider the conduit in its current condition. If corrosion is in progress consider time effects on the estimated probability and report this.

Table A3.14 Factors influencing the likelihood of initiation of erosion into a non-pressurized conduit when the internal condition is not known (IM19).

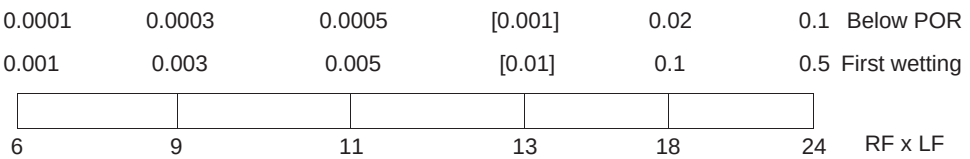
Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Either Conduit type Concrete, masonry, brick OR Steel/Cast Iron Pipe	(3)	Cast in-situ concrete. Precast concrete pipes with high quality joints with water stops. New steel or cast iron with extensive corrosion protection. Concrete encased steel or iron.	Concrete pipe with bell and spigot joints. "New" cast iron or steel pipe ("New" <20 years cast iron, 10 years steel)	Precast concrete culverts Cast in-situ concrete culvert with poor or no water stops "Old" cast iron or steel pipe, not encased ("old" >60 years cast iron, 30 years steel) Corrugated metal pipe <5 years	Masonry, brick. "Old" corrugated metal pipe (>10 years).
Conduit operation	(2)	Dry conduit, inspected regularly	Wet conduit, with low velocity (<1.5 m/sec) flows, dewatered regularly for inspection, or dry	Wet conduit, medium to high velocity flows	Wet conduit, high velocity (>3 m/sec) or surging flows

(*Continued*)

Table A3.14 Continued.

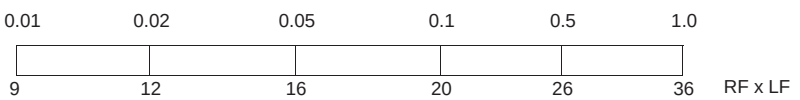
Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Settlement of the conduit	(1)	Rock foundation or measured small settlements	Shallow compressible foundation soils, calculated small settlements	Deep compressible foundation soils calculated large settlements	Measured significant settlement and differential settlement; junction of shaft and conduit within the embankment

Table A3.15 Probability of initiation of erosion into a conduit versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)).



Note: "POR" refers to the Pool of Record level + 0.3 m.

Table A3.16 Probability of the development of piping along the conduit given erosion initiates into the conduit versus $\sum$(Relative importance factor (RF)) x (Likelihood factor (LF)) from Table A3.11.



Note: "POR" refers to the Pool of Record level + 0.3 m.

A3.4 POORLY COMPACTED OR HIGH PERMEABILITY ZONE OR GAP ASSOCIATED WITH A SPILLWAY OR ABUTMENT WALL

A3.4.1 Approach

- Assess the probability of a poorly compacted or high permeability zone or gap for each of the three mechanisms for a spillway or abutment wall;
  - (1) Poorly compacted or high permeability zone associated with the wall (using Table A3.17 in Section A3.4.2).
  - (2) Crack/gap adjacent to the wall (using Table A3.19 in Section A3.4.3).
  - (3) Differential settlement adjacent to the wall (using Table A3.21 in Section A3.4.4)

- Adopt the maximum probability of the three mechanisms.
- For mechanisms (2) and (3) estimate the probability erosion will initiate in the crack or gap using the methods detailed in Section A2.4.
- For mechanism (1) estimate the probability erosion will initiate in the high permeability zone using the methods detailed in Section A3.6.

A3.4.2 Poorly compacted or high permeability zone associated with a spillway or abutment wall (IM20)

Table A3.17 Factors influencing the likelihood of a poorly compacted or high permeability zone associated with a spillway or abutment wall (IM20).

	Relative	*Likelihood Factor (LF)*			
Factor	*Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Compaction of earthfill adjacent to the wall	(3)	Compaction by rollers to >98% standard maximum dry density at OWC−2% to OWC+2%	Compaction by hand and mechanical equipment to >95% standard maximum dry density at OWC−2% to OWC+2%	Compaction by hand equipment, thick layers, dry of optimum moisture content	No formal compaction, or poor compaction practices (e.g. placed in very thick lifts or allowing segregation against wall)
Concrete buttresses	(2)	None	Single but with good compaction around the buttress	Single with poor details such as vertical sides, little evidence of good compaction	Several close together preventing good compaction
Finish on wall	(1)	Smooth planar coupled with flat slope (flatter than 0.5H:1V)	Smooth, planar	Rough and irregular	Vertical and horizontal steps (e.g. masonry/ brick walls)

Table A3.18 Probability of a high permeability zone associated with a spillway or abutment wall versus ∑(Relative importance factor (RF)) x (Likelihood factor(LF)).

0.00005	0.0001	0.0002	[0.0003]	0.002	0.02	Below POR
0.001	0.002	0.005	[0.01]	0.05	0.5	Above POR



6	9	11	13	18	24	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

A3.4.3 Crack/Gap Adjacent to a Spillway or Abutment Wall (IM21)

Table A3.19 Factors influencing the likelihood of a crack or gap adjacent to a spillway or abutment wall (IM21).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Slope of wall	(3)	Sloping, flatter than 0.5H to 1V	Sloping, 0.1H to 1V to 0.5H to 1V	Vertical, or Vertical with flatter slope on lower part	Overhanging over the core width.
Wall type, wall stiffness	(2)	Very stiff gravity wall or counterfort wall	Thin gravity wall Counterfort wall subjected to cyclic reservoir level conditions	Cantilever wall not normally subjected to cyclic reservoir level conditions	Cantilever wall, subject to cyclic reservoir level conditions. Poor freeze/ thaw details
Concrete buttresses	(1)	Single or multiple with good details and proven good compaction around the buttress	None or Single with probably good to reasonable compaction around the buttress	Single with poor details such as vertical sides, little evidence of good compaction	Several close together preventing good compaction

Notes: (a) See Figure A3.2(a).
(b) See Figure A3.2(b).
(c) Select the factor which best describes the worse of the two conditions.

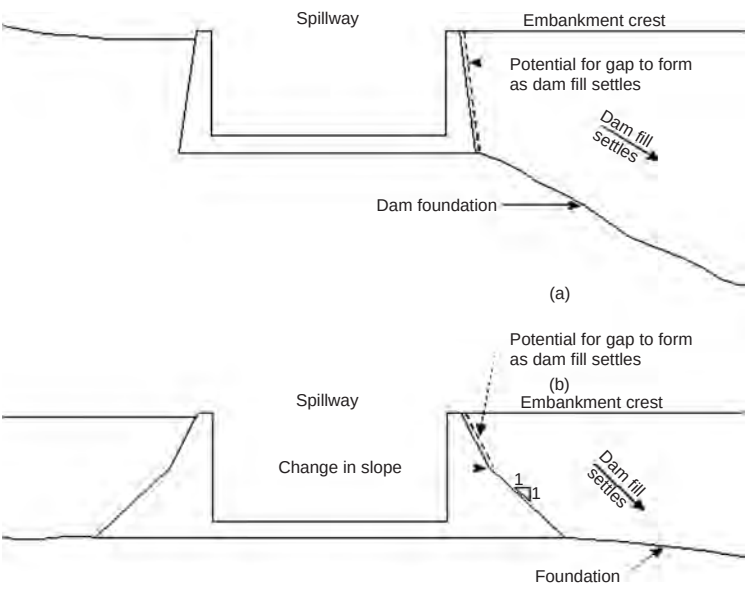


Figure A3.2 Situations where a gap may form between the dam fill and spillway wall (a) Steep foundation adjacent spillway wall; (b) Change in slope of the retaining wall (Fell et al 2004).

Table A3.20 Probability of a gap or crack associated with a spillway or abutment wall versus ∑(Relative importance factor (RF)) x (Likelihood factor(LF)).

0.00005	0.0001	0.0002	[0.0004]	0.002	0.02	Below POR
0.001	0.002	0.005	[0.01]	0.05	0.3	Above POR
6	9	11	13	18	24	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

A3.4.4 Differential settlement adjacent to a spillway or abutment wall (IM22)

Table A3.21 Factors influencing the likelihood of cracking or hydraulic fracturing due to differential settlement adjacent a spillway or abutment wall (IM22).

		Likelihood Factor (LF)			
Factor	*Relative Importance Factor (RF)*	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Cross valley profile under embankment core[(1)]	(3)	Wide bench $W_b/H_w > 2.5$ Note. If this condition is present, probability = zero	Bench adjacent to wall $1.0 < W_b/H_w < 2.5$	Narrow bench adjacent to wall $0.25 < W_b/H_w < 1.0$	No or very narrow bench adjacent to wall. $W_b/H_w < 0.25$
Slope of abutments under embankment core[(1)]	(2)	Gentle abutment slope $\beta_1 < 25°$ Note. If this condition is present, probability = zero	Moderate abutment slopes $25° < \beta_1 < 45°$	Steep abutments $45° < \beta_1 < 60°$	Very steep abutments, $\beta_1 > 60°$
Height of embankment	(1)	Dams less than 15 m high	Dams 15 m to 30 m high	High dams 30 m to 60 m	Very high dams >60 m

Notes: (a) See Figure A2.2 for definitions of W_b, H_w, β_1.
(b) This mechanism only applies for $W_b/H_w < 2.5$.

Table A3.22 Probability of cracking or hydraulic fracture due to differential settlement adjacent a spillway or abutment wall versus ∑(Relative importance factor (RF)) x (Likelihood factor (LF)).

negligible	negligible	negligible	negligible [0.0005]	0.002	0.02	Below POR
negligible	negligible	negligible	negligible [0.005]	0.02	0.2	Above POR
6	9	11	13 14	19	24	RF x LF

Note: "POR" refers to the Pool of Record level + 0.3 m.

A3.4.5 Wrap around details for connection of embankment dam to concrete gravity dam (IM23)

Figure A3.3 shows typical details of the connection of an embankment dam to a concrete gravity dam. There is a potential seepage path along PQRS. The path length and hence the gradient varies with the reservoir level.

There is a potential for the embankment to move away from the concrete dam at PQ and particularly at RS due to settlement of the embankment dam during construction, and usually to a lesser extent after construction. It is recommended that this situation be assessed as follows:

- The primary control on seepage and initiation of erosion is considered to be along QR. The likelihood of a crack or gap being present, and if so, the probability erosion will initiate should be assessed as for an embankment abutting a very stiff retaining wall (refer to Section A3.4.3).
- When assessing the seepage gradient along QR, the likelihood there will be gaps or poorly compacted soil along PQ and RS should be assessed taking account of the factors in Table A3.23. For the worst scenarios, there will be no benefit from the seepage path on PQ and RS if there is a gap there which can be seen on inspection.
- The effect is likely to be most important for reservoir level stages nearing dam crest level.

Table A3.23 Factors to be considered in assessing seepage gradients on wrap-around.

Factors which make it likely there is a good seepage contact along PQ and RS, and gradients will be lower[1]	*Factors which make it likely there is a poor seepage contact along PQ and RS, and gradients will be higher*[1]
Uniform slope with no overhangs	Overhangs in the concrete
Well compacted embankment shoulders or zoning with all materials having a similar and high modulus	Poorly compacted shoulders leading to large settlements during and post construction
Uniform concrete slopes with at least 0.1 H to 1V slope	Change in slope of concrete such as shown in Figure A3.2 allowing a gap to form as the embankment settles
Low embankments	High embankment
Reservoir level at least 6 m below dam crest level so there is a lesser likelihood a crack will persist to reservoir level	Reservoir level approaching dam crest level

Note. (1) See Figure A3.3 for definition of terms.

A3.5 PROBABILITY OF POORLY COMPACTED OR HIGH PERMEABILITY ZONE – FACTORS TO ACCOUNT FOR OBSERVATIONS

Observation of seepage factor

Where there are observations of seepage for the dam these can be used to modify the results of Sections A3.2, A3.3 and A3.4. The probabilities obtained from Sections A3.2, A3.3 and A3.4 are multiplied by the factor from Table A3.24. The multiplication factor

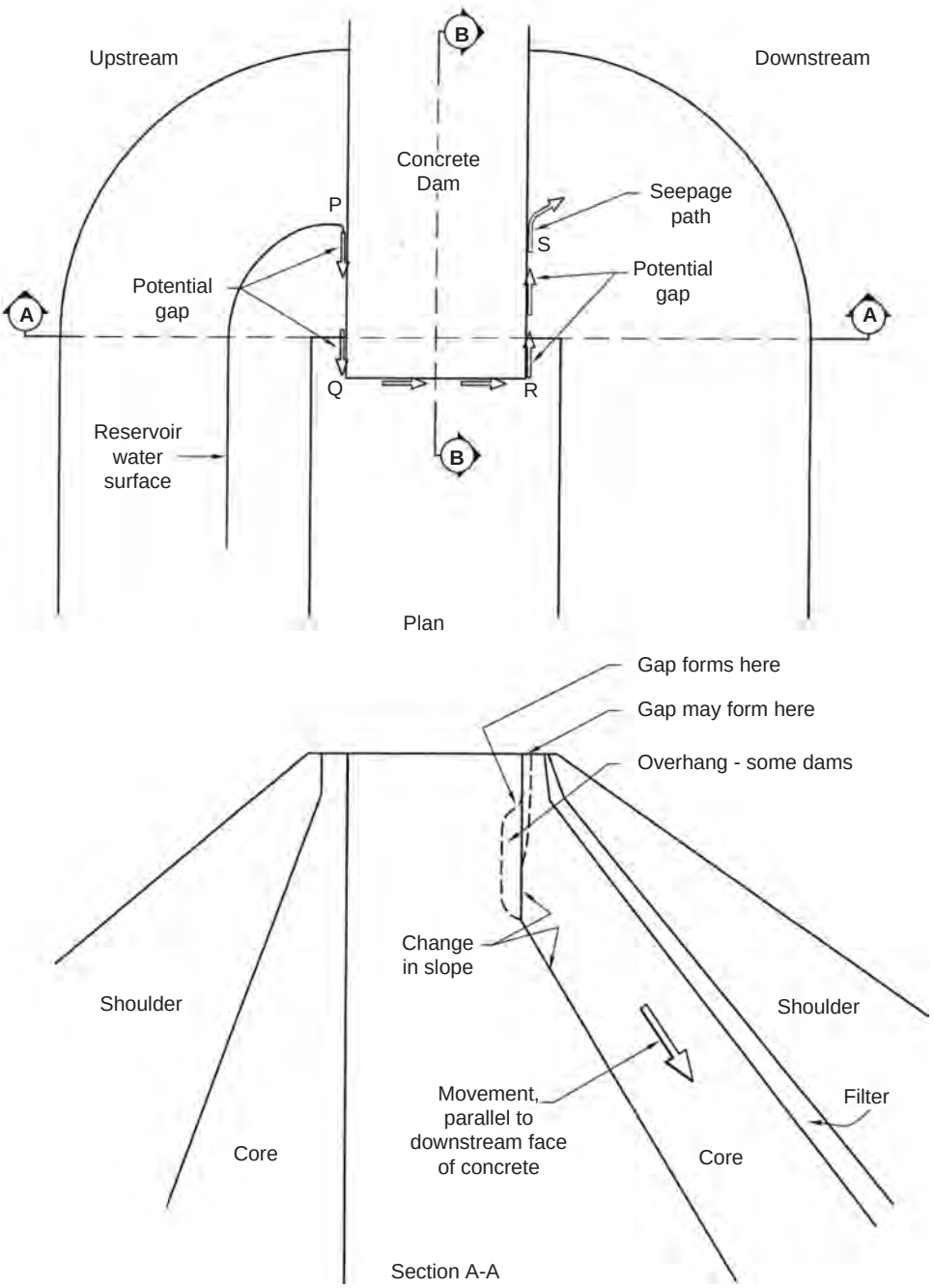


Figure A3.3 Wrap around details for connection of embankment dam to concrete gravity dam.

Table A3.24 Seepage observation factors.

	Influence on Likelihood			
Factor	*Less Likely (1)*	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Observed Seepage	No seepage observed for dams where there is no potential for seepage to be hidden, careful inspection for seepage	Seepage observed at toe of dams with permeable downstream zone or internal drainage systems, or potential for hidden seepage	Wet areas on the downstream slope	Concentrated seepage is present on the downstream slope
Seepage Adjustment Factor (A)	Multiplier = 0.9 to 0.5	Multiplier = 1.0	Probability of high permeability zone = 0.5 to 1.0	Probability of high permeability zone = 1.0
Observations in drill holes/CPT in the core	Multiple drill holes or CPT tests indicate no evidence of high permeability or softened zones	No drill holes or CPT tests in core	Drill holes or CPT tests indicate softened zones in the core	Multiple drill holes or CPT tests indicate persistent high permeability zones are likely to be present
Drill hole/CPT Adjustment Factor (B)	Multiplier = 0.1 to 0.5 depending on quantity and quality of the investigations	Multiplier = 1.0	Multiplier = 5 to 10	Probability of high permeability zone = 0.5 to 1.0
Drilling/Grouting Practices Inducing Defects in the Core		No water drilling or pressure grouting through the core	Drilling through the core with water causing excessive water losses in the core	High pressure grouting was carried out through the core
Adjustment Factors for Induced Core Defects (C)		Multiplier = 1.0	Multiplier = 2 to 5	Multiplier = 5 to 10

Notes: (1) Apply either Seepage Factor (A), Drill hole/CPT Factor (B) or Induced Defect Factor (C), whichever gives greatest probability of high permeability zones.
(2) Use multipliers towards the lower of the range for reservoir level below POR and towards the upper of the range for reservoir levels above POR.

should be selected taking account of what data is available, allowing for the relative importance of the observations.

This factor applies to the assessment of poorly compacted or high permeability zones in the embankment, around conduits and adjacent walls.

A3.6 ESTIMATION OF THE PROBABILITY OF INITIATION OF EROSION IN A POORLY COMPACTED OR HIGH PERMEABILITY LAYER IN THE EMBANKMENT, ADJACENT A WALL OR AROUND A CONDUIT

A3.6.1 Screening check on soil classification

Is the soil cohesive or cohesionless?

If the soil is cohesionless or has a Plasticity Index ≤ 7, follow the procedures in Sections A3.6.2 and A3.6.3 to assess the probability of backward erosion and suffusion.

If the soil is cohesive with a Plasticity Index >7, then the likelihood of backwards erosion and suffusion is negligible under the seepage gradients which occur in a conventional dam. Consider if erosion may occur through a crack in the soil using the procedure detailed in Sections A3.6.4 and A3.6.5. If the seepage gradients are >4, consider suffusion for soils with a Plasticity Index ≤ 12.

A3.6.2 Assessment of The Probability of Initiation of Backward Erosion in a Layer of Cohesionless Soil or Soil with Plasticity Index ≤ 7

The steps to be followed are:

- Estimate the average seepage gradient (i_{av}) through the core at the level of the high permeability layer for the reservoir stage under consideration.
- Estimate the time it will take to develop a seepage gradient in the layer from the estimated permeability of the soil, allowing for potential collapse of the layer on saturation if the soil is not well compacted. Use Table A3.25 to assist in this estimate. From this and the time the reservoir will be above this stage assess whether there is sufficient time to develop the seepage gradient in the layer. For layers below the normal operating pool level the layer will be saturated and the seepage gradient will develop as the reservoir rises.

Table A3.25 Time to develop seepage gradient in cohesionless soils.

Method of compaction (see Tables 6.1 and 6.2 for detailed descriptions	*Time for developing seepage gradient with collapse settlement*	*Time for developing seepage gradient if there is no collapse settlement*
No formal compaction	Minutes	Not applicable. Collapse settlement is highly likely
Tracking by dozer or rolled in layers too thick for the equipment	Minutes to a few hours	Not applicable. Collapse settlement is highly likely
Compacted by rollers in suitable layer thicknesses to normal compaction standards	Not applicable	Hours to days for silty sands and sands.
Compacted by rollers in suitable layer thicknesses to normal compaction standards with well documented compaction records	Not applicable	Hours to days for silty sands and sands.

- If the time to develop seepage gradient in the layer use se the methods described in Chapter 8, Section 8.4 to assess the likelihood of backward erosion piping. Do not use the Piping Toolbox methods

A3.6.3 Probability of initiation of erosion by suffusion in a layer of cohesionless soil or soil with plasticity index ≤7 (PI ≤12 for Seepage Gradients >4)

Check if the soil is potentially internally unstable

Check if the proportion of the finer fraction is less than 40% of the total mass of the soil. The finer fraction is defined by the point of inflection of broadly graded or gap graded soils. This is a check on whether the coarser particles will form a matrix into which the finer particles will fit. If the answer is yes, continue to assess the likelihood of the soil being internally unstable as described in the following steps. If not (there are more than 40% finer particles) the coarse particles will "float" in the finer particles and suffusion is not possible. These soils may experience backward erosion.

Assess the probability the soil is internally unstable (P_{IUS})

From the particle size distribution for the soil, determine the d_{15}, d_{60} and d_{90} sizes (the particle size for which 15%, 60% and 90% are finer). The grading curve is not adjusted for this procedure. Then estimate the probability the soil is internally unstable (P_{IUS}) from Figure A3.4 for soils with more than 10% fines passing 0.075 mm (#200 sieve), and Figure A3.5 for soils with less than 10% fines passing 0.075 mm.

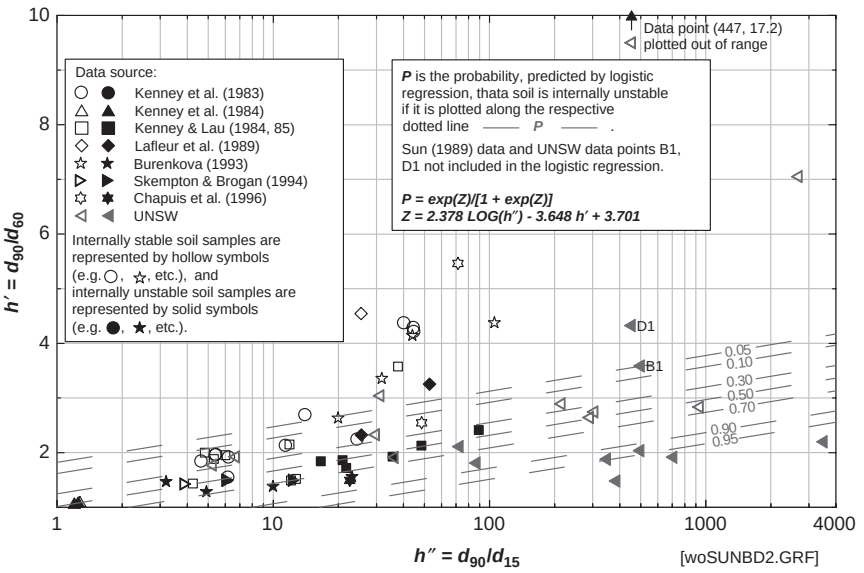


Figure A3.4 Contours of the probability of internal instability for silt-sand-gravel soils and clay-silt-sand-gravel soils of limited clay content and plasticity, Plasticity Index ≤12 (Wan and Fell 2004).

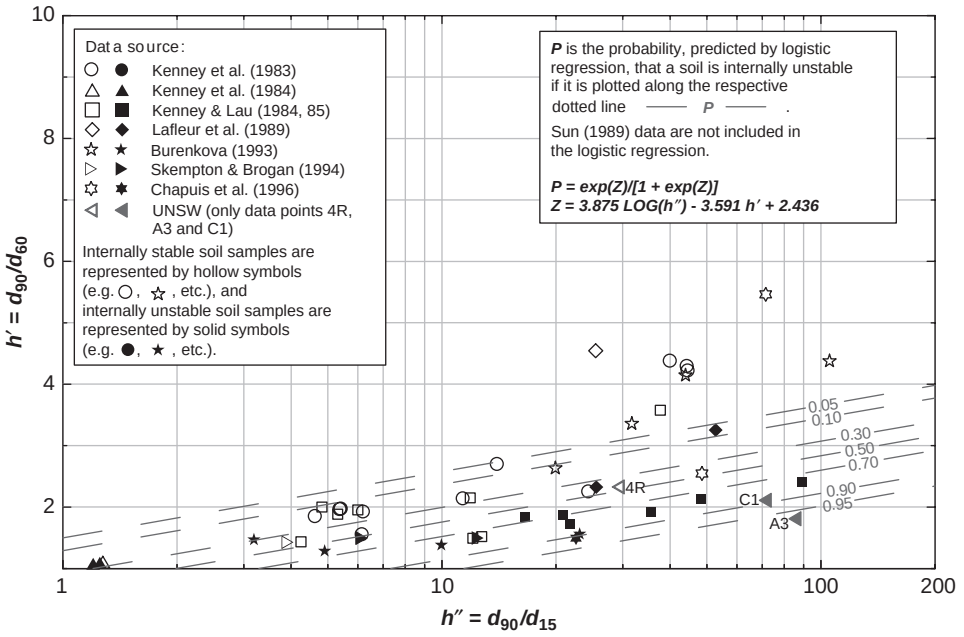

Data source:
Kenney et al. (1983)
Kenney et al. (1984)
Kenney & Lau (1984, 85)
Lafleur et al. (1989)
Burenkova (1993)
Skempton & Brogan (1994)
Chapuis et al. (1996)
UNSW (only data points 4R, A3 and C1)
Internally stable soil samples are represented by hollow symbols (e.g. ○, ☆, etc.), and internally unstable soil samples are represented by solid symbols (e.g. ●, ★, etc.).
P is the probability, predicted by logistic regression, that a soil is internally unstable if it is plotted along the respective dotted line —— P ——.
Sun (1989) data are not included in the logistic regression.
P = exp(Z)/[1 + exp(Z)]
Z = 3.875 LOG(h″) - 3.591 h′ + 2.436
h′ = d90/d60
h″ = d90/d15
0.05
0.10
0.30
0.50
0.70
0.90
0.95
4R
C1
A3
10
8
6
4
2
1
10
100
200

Figure A3.5 Contours of the probability of internal instability for sand-gravel soils with less than 10% non-plastic fines passing 0.075 mm (Wan and Fell 2004).

Assess the probability that given the soil is internally unstable erosion by suffusion will begin (P_{SI})

Assess the probability that given the soil is internally unstable suffusion will begin under the seepage gradient in the highly permeable layer using 3.26. The probability of suffusion and backward erosion should both be assessed and carried forward in the analysis.

Table A3.26 Seepage gradients at which suffusion may occur.

Porosity (volume of voids/total volume of soil)	*Average seepage gradient across embankment core (i)*					
	0.05	*0.1*	*0.25*	*0.5*	*1.0*	*2.0*
<0.20	0.01	0.02	0.05	0.2	0.9	0.99
0.20 to 0.25	0.02	0.05	0.1	0.5	0.95	0.99
0.25 to 0.30	0.05	0.1	0.2	0.9	0.99	0.99
0.30 to 0.35	0.1	0.2	0.5	0.95	0.99	0.99
>0.35	0.2	0.5	0.9	0.99	0.99	0.99

A3.6.4 Probability of initiation of erosion in a poorly compacted or high permeability cohesive soil layer and in silt-sand-gravel soils in which collapse settlement may form a crack or flaw

It is well documented that internal erosion and piping occurs in poorly compacted cohesive soils. This is particularly so for dispersive soils. The mechanism is potentially of two types:

- The soil behaves as a series of clods with openings between the clods in which water passes.
- The soil collapses on saturation forming a crack or flaw in which the water flows. This is most likely where there is poorly compacted soil against a pipe but is possible within layers of soil.

To model this it is most practical to assume a crack is formed and to assess the likelihood of erosion initiating in the crack.

The procedure is:

- Assess the thickness of the layer of soil which is poorly compacted (T_p). There may be a single layer or several layers. The minimum layer thickness adopted should be 300 mm.
- Estimate the amount by which the layer may collapse (C_F) using Table A3.27. Then estimate the height of the gap which could result (G) from $G = (T_p) \times (C_F)$. This represents the scenario with the weight of the soil above being supported on non-collapsed soil adjacent.
- Assume G is the height of the crack which is formed, and use the method outlined in Section A2.4 to estimate the probability of initiation of erosion given this width crack, the average gradient through the core at the level of the high permeability layer, and the soil properties.

The mechanism applies to such zones within the embankment, on the embankment/foundation contact, around conduits, and adjacent to walls.

This method should be also applied to silty sands, and silty sandy gravel soils which may be subject to collapse settlement even if the soils are non plastic, since they will erode rapidly in a crack.

A3.6.5 Probability of initiation of erosion in a poorly compacted or high permeability cohesive soil layer around a conduit

The method assumes that the critical case is where poorly compacted soil surrounding the conduit collapses on saturation and a gap is formed.

The procedure is:

- Assess the thickness of the layer of soil which is poorly compacted (T_p). There may be a single layer or several layers.
- Estimate the amount by which the layer may collapse (C_F) using Table A3.27 as a guide. Then estimate the height of the gap which could result (G) from

Table A3.27 Amount of collapse settlement which may occur on saturation versus compaction properties.

Description of the method and degree of compaction of the core		*Amount of collapse settlement as a proportion of the layer thickness*
Soil placed with, no formal compaction (e.g. by horse and cart in old dams, or by pushing into place by excavator or bulldozer or in very thick layers) No control on layer thickness, Well dry of optimum moisture content.	Layers very poorly compacted, dry of standard optimum moisture content, e.g. <90% standard dry density ratio, 3% dry of standard OWC	0.02 to 0.05
Soil placed and compacted by bulldozer, no compaction by rollers, or rolled in thick layers beyond the capability of the roller Layer thickness at or beyond the limit of compaction equipment Dry of optimum moisture content	Layers poorly compacted, dry of standard optimum moisture content e.g. <93% standard dry density ratio, 2% to 3% dry of standard OWC	0.01 to 0.02
Soil rolled in layers near the limit of the capability of the rollers, at moisture contents dry of standard OWC	Compacted to e.g. 93–95% standard dry density ratio, moisture content 2% to 3% dry of standard OWC	0.005
Soil compacted by suitable rollers in suitable layer thickness Around optimum moisture content	Well compacted to e.g. 95–98% I standard dry density ratio, moisture content 2% dry of optimum to 1% wet of standard OWC	Will not collapse, but for the poorly compacted layer within 0.005
As above but with good documentation and records.	All very well compacted to e.g. ≥98% standard dry density ratio, moisture content 2% dry of optimum to 1% wet of standard OWC	Will not collapse, but for the poorly compacted layer within 0.005

$G = (T_p) \times (C_F)$. To do this take account of the dimensions of the conduit and the trench in which it is placed.

- For cases where it appears that the soil around the pipe is well compacted assume and where crack widths <5 mm are calculated, assume a crack width of 5 mm to allow for possible shrinkage of the soil from the pipe during construction or in service.
- Assume G is the height of the crack which is formed, and use the method outlined in Section A2.4 to estimate the probability of initiation of erosion given this width crack, the average gradient through the core at the level of the high permeability layer, and the soil properties.

A3.6.6 Probability of Initiation of Erosion in a High Permeability Soil Due to Frost Action

The procedure is:

- Assess the width of the frost induced crack or flaw using the procedure detailed below.

Table A3.28 Width of frost induced flaw versus (RFxLF).

(RFxLF) from Table A3.7	*(RFxLF) from Table A3.9*	*Width of flaw-mm*
24	28	20
21	22	10
16	18	5
12	13	2
10	12	Zero

- Use the method outlined in Section A2.4 to estimate the probability of initiation of erosion given this width crack or flaw, the average gradient through the core at the level of the high permeability layer, and the soil properties.

The effects of frost action are complex and include formation of cracks due to heave, and formation of ice lenses. The latter may melt in summer months and leave pathways in which erosion may initiate. Where there is specific information about frost effects for the dam being assessed, that information should be used by the risk analysis team to assess likely defect widths. In the absence of such data Table A3.28 should be used. These widths may be applied to the full depth of penetration of frost from Table A3.8 as the evidence is the ice lenses may be as thick at the base of freezing as at the surface.

A3.7 ALLOWANCE FOR THE PRESENCE OF UNKNOWN AND UNPREDICTABLE FLAWS IN THE CORE OF THE EMBANKMENT

A3.7.1 Background

Even with well designed and constructed dams, for which there is good record of construction, it is possible that there are unknown and unpredictable flaws in the core which may, under some circumstances, lead to initiation of erosion.

In the opinion of the development team, the unknown and unpredictable flaws are most likely to be a poorly compacted layer or a layer that was allowed to crack or soften during construction. For reasons of continuity the flaw is probably more likely to be in the upper part of the dam where the core width is smaller. The flaw could also be due to unpredictable cracking or to a pathway created by installation of instrument cables or tubes.

The likelihood of an unknown flaw is greater above the reservoir pool levels that the dam has been tested. Below the POR there is a greater chance that the flaw has been detected and addressed as a known issue.

The probability of these unknown flaws should be low as the risk analysis has already considered the more predictable modes of initiation. In most cases they should not contribute significantly to the probability of failure because other more predictable flaws will have a greater probability.

If a dam does not have filters or a transition zone (e.g. a homogeneous dam), the probability of failure approaches that of the probability of the presence of a flaw, and

the probability given there is a flaw, erosion initiates. In these cases if the dam has a high consequence of failure, even a very small probability of a flaw can be significant contributor to risk as there are little other defensive measures in place.

The potential for unknown or unpredictable flaws in dams is always a possibility. It is therefore considered important that the responsible dam safety decision makers be aware of this when assessing the safety of the dam.

This section provides advice on what to do in these situations.

The development team has strived to address all failure modes that, in their experience, occur in embankment dams. Procedures are also suggested to handle failure modes not well covered in this methodology. However, in the end, as pointed out by the late Dr. Ralph Peck in his paper "The Risk of the Oddball", there remains the potential that even with the "... efforts of even the most experienced engineers the most significant potential failure mode may occasionally be overlooked." As Peck summarizes, the only recourse is continuing surveillance. Surveillance should always be part of the monitoring program for the dam and all dam owners should be aware of this. The risk analyst should remind the owner of this in the preparation of the report. Armed with possibly a fresh look at potential failure modes, a review of the surveillance program at the end of the risk analysis is always a good idea.

A3.7.2 Recommended Procedure

For dams which do not have filters or transition zones protecting all or part of the core, and the assessed probability of a flaw for the initiation modes described in Sections A2 and A3 give very low probability of a flaw being present ($\leq 10^{-4}$ for reservoir levels above POR and $\leq 10^{-5}$ for reservoir levels below POR) and the consequences of failure are high, it is recommended that the procedure detailed below be followed to check the effect on the risk analysis of the presence of unknown and unpredictable flaws.

A3.7.3 Suggested Size and Location of the Flaw and the Probability of Occurrence

It is suggested that:

- A 2 mm wide flaw is assumed to be present in a location that is critical for each reservoir level under consideration. This can be where the gradient of flow through the flaw will be greatest, where there is no filter or transition zone, or the filter or transition zone is so coarse that there is a significant probability of being in the continuing erosion range.
- Develop a failure path for this scenario.
- Assign a probability of the presence of the flaw of 10^{-4} for reservoir levels above POR and 10^{-5} for reservoir levels below POR.
- Complete the evaluation of the event tree for this failure path.

See the Supporting Document for the rationale behind these suggestions.

A3.7.4 Assessing the Probability of Failure if the Flaw Were to Exist and Reporting the Outcome of the Analysis

Assess the probability of failure for each of the reservoir stages affected by the flaw by assuming the flaw is as described in Section A3.7.3.

Report this separately to the other failure modes. Make the case in the report of how likely (or unlikely) the dam has such a flaw. This discussion could focus on the amount and quality of construction control information, the climatic conditions at the dam site that may have been present during construction, and other factors considered relevant.

Whether or not this probability of failure is combined with the other failure modes to assess the overall probability of failure is for the Agency for whom the risk analysis is being carried out to decide.

A4 Probability of Initiation of Erosion in a Soil Foundation

A4.1 SCREENING CHECK ON SOIL CLASSIFICATION

Is the soil cohesive or cohesionless?

- **If the soil is cohesionless** or has a Plasticity Index ≤ 7, then follow the procedures in use the methods described in Section A4.2 to assess the likelihood of backward erosion piping. Do not use the Piping Toolbox.
- Toolbox methods. Use the method described in Section A3.6.3 of this Appendix to assess the likelihood of suffusion.
- **If the soil is cohesive with a Plasticity Index >7**, then the likelihood of backwards erosion and suffusion is zero under the seepage gradients which occur in foundations of a conventional dam. Consider if erosion may occur through a crack in the soil using the procedure detailed in Section A4.2. If the seepage gradients are >4, consider suffusion for soils with a Plasticity Index ≤ 12.

A4.2 ASSESSMENT OF THE PROBABILITY OF INITIATION OF BACKWARD EROSION PIPING IN A LAYER OF COHESIONLESS SOIL OR SOIL WITH PLASTICITY INDEX ≤ 7 IN THE FOUNDATION (IM24)

The steps to be followed are:

a) Estimate the probability (P_{cl}) there is a continuous layer of cohesionless soil or soil with PI ≤ 7 from upstream of the embankment, to downstream of the embankment. The layer does not have to be exposed to the ground surface downstream of the embankment. That is, it may be overlain by a layer of cohesive soil.
b) If there is no continuous layer such as shown in Figure A4.1, then backward erosion is not possible and the probability of backward erosion may be taken as

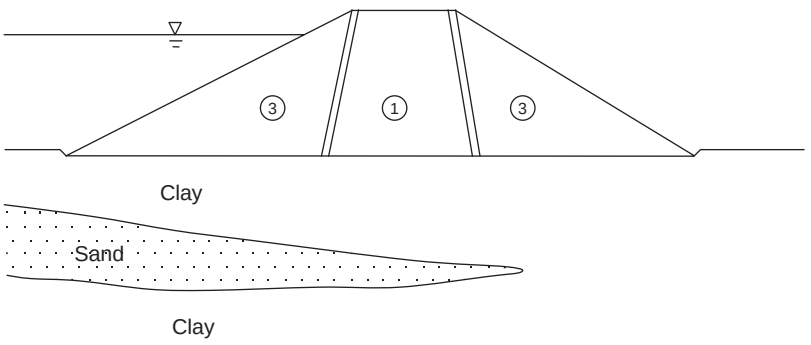


Figure A4.1 An example of a situation where there is no continuous layer of cohesionless soil in the foundation and backward erosion cannot occur.

zero. In some situations there will be some uncertainty and the probability of a continuous layer should be assessed from the geotechnical borehole data, understanding of the depositional conditions in which the soils were and deposited, piezometer data, including response of the piezometers to changes in reservoir level. P_{cl} is likely to be between 0.1 and 1.0 for most situations

c) Assess the probability of "heave" (P_H) or reaching the critical gradient (i_{cr}) which is a zero effective stress condition, at the toe of the embankment for the reservoir stage under consideration. This assessment should consider the potential for localized areas of a thinner confining layer within a distance of 2 × dam height downstream of the dam toe (e.g. a toe ditch). This should be done using local gradients at the toe of the embankment from piezometer readings, and/or finite element seepage flow modeling.

d) Estimate the probability of initiation and progression of backward erosion given heave has occurred (P_{IH}) using Chapter 8, Section 8.4.Do not use the Piping Toolbox. For situations where sand boils have been observed, the probability of initiation and progression for the reservoir level at which sand boils have been observed and above is assumed to be 1.0

e) Given that backwards erosion may initiate and progress without a heave condition being present, estimate the probability of initiation and progression of backward erosion given heave has not occurred (P_{INH}) using Chapter 8, Section 8.4.

f) Estimate the probability of initiation and progression of backward erosion (P_I) from the results of the assessments above. $(P_I) = (P_{cl}) \times [(P_H) \times (P_{IH}) + (1 - (P_H) \times (P_{INH})]$.

A4.3 ESTIMATION OF THE PROBABILITY OF INITIATION OF SUFFUSION IN A COHESIONLESS LAYER IN THE FOUNDATION (IM25)

First Estimate the probability (P_{cl}) there is a continuous layer of cohesionless soil from upstream of the embankment, to downstream of the embankment.

Second Use the method outlined in Section A3.6.3 to estimate the probability of suffusion. Multiply this value with the probability there is a continuous layer subject to suffusion in the foundation.

A4.4 ESTIMATION OF THE PROBABILITY OF INITIATION OF EROSION IN A CRACK IN COHESIVE SOIL IN THE FOUNDATION (IM26)

A4.4.1 Overall approach

First: Estimate the probability (P_{cl}) there is a layer of soil beneath the embankment in which a continuous crack or interconnected pattern of cracks may exist. Cracking may be the result of differential settlement in the foundation, or desiccation cracks in the foundation soil which was not stripped from the foundation. The question is best put in terms of the width of the crack. For example "what is the probability of a continuous crack or pattern of cracks 5 mm wide". It may be useful to consider the question for 1 mm, 5 mm, and say 10 mm wide cracks, as there may be a significantly high likelihood of narrow cracks, and much lower likelihood of wider cracks.

Second: Estimate the probability erosion will initiate in the cracks (P_{IC}) using Section A2.4.

The probability of initiation = $(P_{cl}) \times (P_{IC})$.

A4.4.2 Some Factors to Consider in this Assessment and Suggested Method for Estimating the Probability of a Continuous Crack

(a) For Cracking Due to Differential Settlement

The situations which are likely to result in cracking in the foundation soils are essentially the same as those causing cracking low in the embankment due to differential settlement in the foundation. An example is shown in Figure A3.5(b). Given this, it is suggested that if in using Section A3.2.3 to assess the probability of cracking in the embankment due to differential settlement in the foundation, the presence of cracking in the embankment is likely to be on the foundation/embankment contact, then this probability also be assigned to the probability of cracking in the cohesive soil in the foundation beneath the embankment.

If the assessment of cracking in Section A3.2.3 is for features such as those in Figure A3.5(a) and (c) which will cause cracking near the crest of the dam, then cracking in the foundation due to differential settlement may be assumed to be negligible.

(b) For Cracking Due to Desiccation

Some factors to consider in assessing the likelihood of continuous or interconnected cracking include:

- Whether the soils in the foundation on which the dam was built are susceptible to desiccation in the climatic conditions at the site. This can be assessed using

Section A2.2.5 and Tables A2.11 and 2.12 considering the foundation soil as the "core material".
- Whether desiccation cracking is evident in the soil surrounding the dam.
- Whether the cracking is likely to persist below the level of the cut-off for the dam. This can be assessed using observed desiccation crack depths for the site, or using Tables A2.24 and A2.25 to estimate the depth and width of cracking, and knowing the depth of the cut-off below the ground surface.
- The continuity is likely to be linked to the width of the cut-off, with wide cut-offs being less likely to have continuous cracking than narrow cut-offs.

A5 Probability of the Presence of Open or Infilled Defects in Rock Foundations and their Effect on the Likelihood of Internal Erosion and Piping in the Foundation

The framework in the Piping Toolbox considers the geological processes which can lead to the formation of open or in filled defects or solution features in rock. The geological processes considered are:

- Defects related to stress relief effects in valley sides (Section A8.2 of the Toolbox).
- Defects related to stress relief effects in the valley floor – valley bulge and rebound (Section A8.3 of the Toolbox).
- Solution features for rock subject to solution such as limestone, dolomite, gypsum and salt (Section A8.4 of the Toolbox).
- Defects associated with landslides and faults and shear zones (Section A8.5 of the Toolbox)

An outline of the overall approach is summarized in the flow chart shown in Figure A5.1.

The steps to assess the probability such defects and solution features exist in the dam foundation below the core are as follows;

- Assess the probability of one or more continuous in filled or open defects or solution features in the rock in the foundation beneath the core of the embankment for each of the types of geological features.

  For each type of geological feature, there are two parts to the assessment; the first is based on the geology and topography of the dam site. This information will be available for all dam sites. The second is based on observations and site investigation data of which there will be greatly varying amounts for different dams. It is anticipated that for some older dams there may be virtually no such data.

  The relative importance factors are selected taking account of the data which is likely to be available. While continuity of these features is very important it is unlikely there will be good quality information on this for many dams.
- Assess the width of the open or in filled defects or solution features, how far these occur into the foundation, and how their widths vary with depth in the foundation.
- Assess the probability that the defects are in filled.

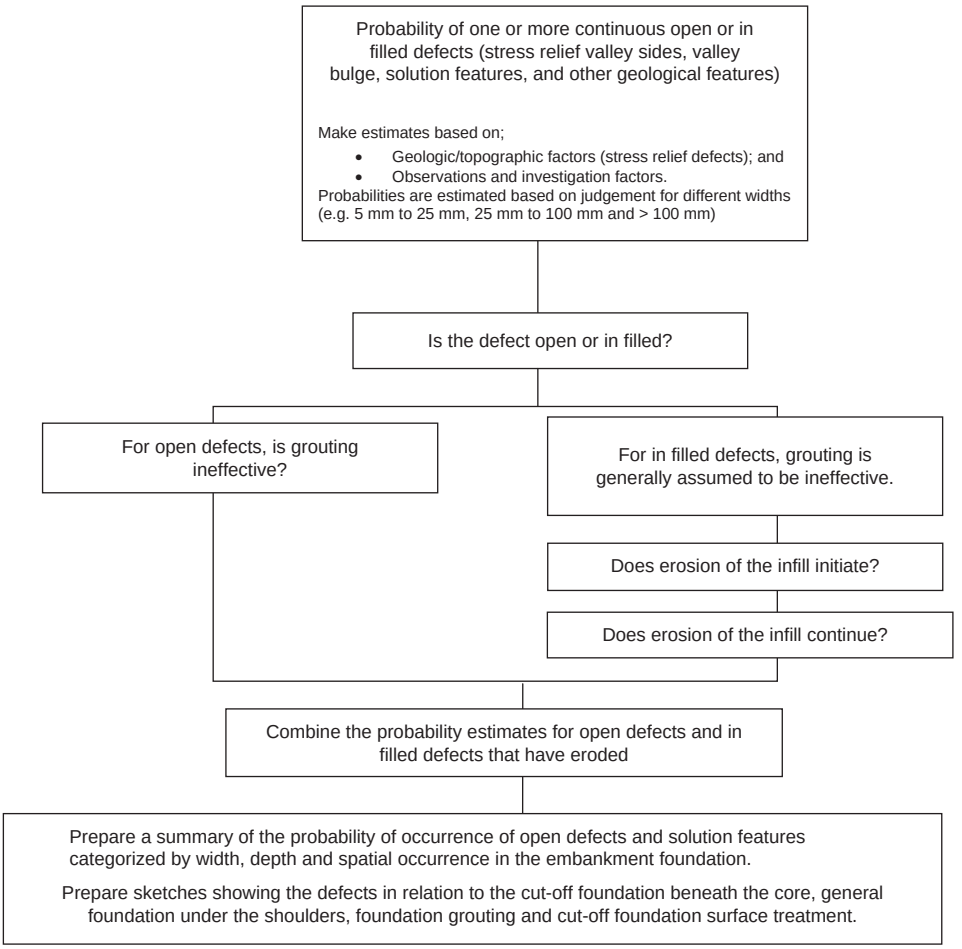

Probability of one or more continuous open or in filled defects (stress relief valley sides, valley bulge, solution features, and other geological features)
Make estimates based on;
Geologic/topographic factors (stress relief defects); and
Observations and investigation factors.
Probabilities are estimated based on judgement for different widths (e.g. 5 mm to 25 mm, 25 mm to 100 mm and > 100 mm)
Is the defect open or in filled?
For open defects, is grouting ineffective?
For in filled defects, grouting is generally assumed to be ineffective.
Does erosion of the infill initiate?
Does erosion of the infill continue?
Combine the probability estimates for open defects and in filled defects that have eroded
Prepare a summary of the probability of occurrence of open defects and solution features categorized by width, depth and spatial occurrence in the embankment foundation.
Prepare sketches showing the defects in relation to the cut-off foundation beneath the core, general foundation under the shoulders, foundation grouting and cut-off foundation surface treatment.

Figure A5.1 Flow chart for estimating the probability, width, depth and spatial distribution of continuous open defects and solution features in rock foundations.

- Assess the probability that grouting has not been effective in cutting off the open defects. For in filled defects, it is assumed that grouting is not effective.
- Assess the probability of cut-off walls not being effective in cutting off open or in filled defects or solution features.
- Assess the probability that erosion of the infill will initiate given grouting and cut-off walls are not effective.
- Assess the probability that erosion of the infill in defects or solution features will continue.
- Combine the probabilities for open defects and in filled defects which will potentially erode.
- Describe the open and in filled defects and solution features, their width, depth, spatial distribution in the foundation, and how these relate to the cut-off and

general foundation of the embankment beneath the core. In particular identify features which will be in contact with the core at the base of the cut-off and in the sides of the cut-off trench.

Given the defects or solution features occur below the core of the embankment, identify the potential failure paths, and potential breach modes due to the open defects and solution features. Possible failure paths are;

a) Gross enlargement of the defect – this can release the storage but is usually not capable of breaching the dam unless the feature is large relative to the size of the dam
b) Slope instability of the embankment due to increased pore pressures caused by seepage up into the downstream shell;
c) Unraveling of the embankment due to seepage exiting into the downstream shell;
d) Initiation of internal erosion of the embankment at or into the foundation by backward erosion piping or scour followed by gross enlargement, slope instability, unraveling or sinkhole development in the embankment.

This is the end product of what is covered in this Section.
Then for each potential failure path and breach mechanism, assess:

a) The probability of initiation of erosion of the core into the open defect or scour of the core at the defect-core interface.
b) The probability of progression.
c) The probability of detection, intervention and repair.
d) The probability of breach.

Tables have been produced in the Piping Toolbox to evaluate the probabilities for assessing the presence of continuous open or in filled defects and solution features caused by these geological processes. While the principles of these tables are reasonable, experience in the use of these tables is that they over-estimate probabilities so in practice the tables should not be used and expert judgment must be used based on the available geological information.

A6 Probability of Initiation of Erosion from Embankment into Foundation

For erosion to initiate from the embankment into the foundation, requires open joints in rock or coarse soils in the base or sides of the cut off trench. For cases where a cut off trench is not present, then the issue is whether erosion can occur along the core-foundation contact.

Internal erosion may initiate by:

- Backward erosion or suffusion in a high permeability zone in the core or cut-off trench.

- Scour of the core at the core – foundation contact by water flowing in joints in the rock foundation.
- Erosion in a crack or hydraulic fracture across the cut off trench.

The method to assess the probability of initiation of erosion into these features is:

(a) **Assess from the available data the probability of a continuous pathway of open joints in rock or coarse grained soils (P_{path}) in the base or sides of the core trench or core-foundation contact.** In many cases it will become apparent at this stage that there is little or no likelihood of such features being present and the probability of piping into the foundation may be assessed as negligible. Refer to Section A9.3 of the Piping Toolbox for guidance on estimating the probability.

(b) **Assess the probability of the initiation of internal erosion by backward erosion or suffusion (P_e) starting at the core-foundation contact given there is a continuous path.** Refer to Section A9.5 of the Piping Toolbox for guidance on estimating the probabilities.

(c) **Assess the probability of initiation of scour.** Refer to Section A9.6 of the Piping Toolbox for guidance on estimating the probabilities.

(d) **Assess the probability of erosion of the core following hydraulic fracture due to arching in a narrow core trench**. Refer to Section A9.7 of the Piping Toolbox for guidance on estimating the probabilities.

A7 Probability of Initiation of Scour at the Core-Foundation Contact

This considers the likelihood that seepage flows within a continuous pathway in a rock or soil foundation may initiate erosion of the core material at the core-foundation contact.

The steps in the assessment are as follows;

- Estimate the probability of a continuous pathway for erosion at the core-foundation contact (P_{path}) from Section A5.
- Estimate the probability of erosion of the core material at the core-foundation contact (P_{ic}) using the method for erosion in a crack in the core as described in Section A2.4. The hydraulic gradient used in the assessment should be based on the estimated seepage gradient on the core-foundation contact. Allowance should be made for varying crack widths as shown in Figure A7.1. Guidance is given in Section A9.6.2 of the Piping Toolbox. Assume that the hydraulic shear stresses imposed on the core by the water flowing in the open joints is equivalent to those for an equivalent crack width.
- The probability of internal erosion by scour $= P_{path} \times P_{ic}$.
- Do this for each of the potential defect openings and soil particle sizes and take these estimates forward into the assessment of continuation, progression, detection, intervention and repair, and breach.

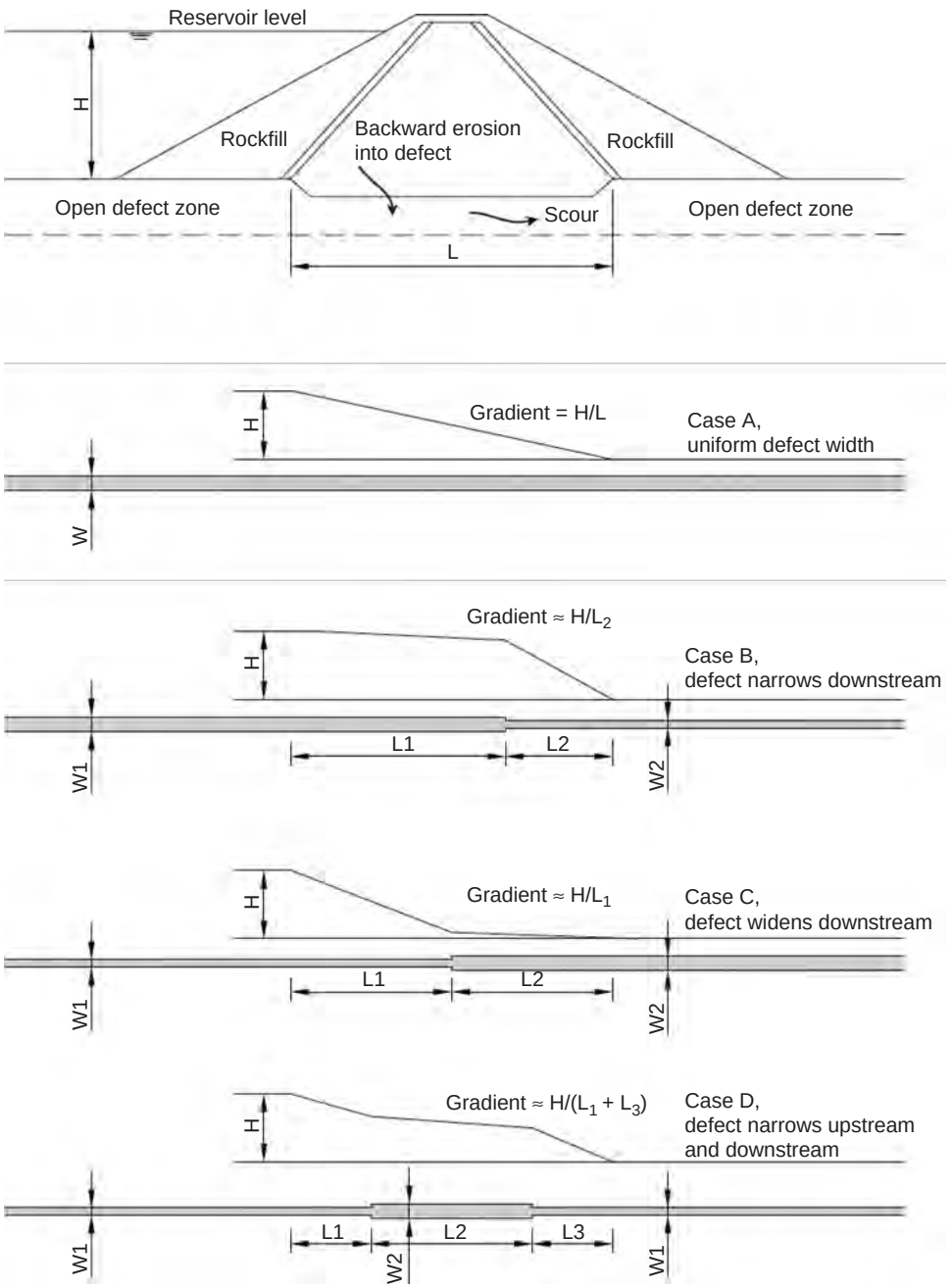


Figure A7.1 Examples of foundations with continuous open defects of varying width.

A8 Probability of Continuation

A8.1 PROBABILITY OF CONTINUATION FOR INTERNAL EROSION IN THE EMBANKMENT

A8.1.1 Internal erosion through the embankment – overall approach

Step 1: Assess which of the following five scenarios is most applicable to the dam section and failure path that is under consideration:

- Scenario 1: Homogeneous zoning with no fully intercepting filter.
- Scenario 2: Downstream shoulder of fine grained cohesive material which is capable of holding a crack/pipe. Soils which are capable of holding a crack or pipe are:
  - well compacted shoulder (shell), containing >5% plastic fines; or
  - poorly compacted shoulder (shell), containing >15% plastic fines
  - well compacted shoulder, containing >30% non plastic fines
  - poorly compacted shoulder, >30% non plastic
- Scenario 3: Filter/transition zone is present downstream of the core or a downstream shoulder zone which is not capable of holding a crack/pipe. This includes earthfill dams with a chimney filter.
- Scenario 4: Erosion into a crack or open joint (e.g. open joint or crack in a conduit or adjoining concrete structure).
- Scenario 5: Erosion into a toe drain.

Step 2: For Scenarios 1 and 2, estimate the probability for Continuing Erosion based on the guidance given in the applicable section (Sections A8.1.2 or A8.1.3).

For Scenario 3, a four way split for filtering behavior is recommended in the event trees:

- **Seals with No Erosion** – the filtering material stops erosion with no or very little erosion of the material it is protecting. The increase in leakage flows is so small that it is unlikely to be detectable. The No Erosion branch on the event tree is a "No Breach" branch.
- **Seals with Some Erosion** – the filtering materials initially allow erosion from the soil it is protecting, but it eventually seals up and stops erosion. Leakage flows due to piping can be up to 100 l/s, but are self healing.
- **Seals with Excessive Erosion** – the filter material allows erosion from the material it is protecting, and in the process permits large increases in leakage flow up to 1000 l/s, but the flows are self healing. The extent of erosion is sufficient to cause sinkholes on the crest and erosion tunnels through the core.
- **Continuing Erosion** – the filtering material is too coarse to stop erosion of the material it is protecting and continuing erosion is permitted. Unlimited erosion and leakage flows are likely.

For Scenario 3, estimate the probability for Some Erosion, Excessive Erosion and Continuing Erosion based on the guidance given in Section A8.1.4.

For Scenario 4, estimate the probability for Some Erosion, Excessive Erosion and Continuing Erosion based on the guidance given in Section A8.1.5.

The maximum leakage flows for the various filter erosion categories described above are used to assess the potential for unraveling and instability in the Breach Mechanism phase.

A8.1.2 Probability for Continuation – Scenario 1 (Homogeneous zoning)

There is no potential for filtering action for this scenario. Adopt a probability of Continuing Erosion = 1.0.

A8.1.3 Probability for Continuation – Scenario 2 (Downstream Shoulder can hold a Crack or Pipe)

The issue for this scenario is whether the crack/high permeability feature that is present through the core is continuous through the downstream shoulder, or if not, whether it can find an exit. This depends on the following factors:

- The mechanism causing the concentrated leak, in particular whether it also causes cracking in the shoulder.
- The material characteristics and width of the downstream shoulder zone.

Use Table A8.1 to evaluate the conditional probability of continuing erosion.

A8.1.4 Probability for Continuation – Scenario 3 (Filter/transition zone is Present Downstream of the Core or a Downstream Shoulder Zone which is not Capable of Holding a Crack/pipe)

The method of assessing the probability for continuation depends on the information that is available on the particle size distributions of the core and filter/transition/shoulder materials. The two approaches are as follows:

- If particle size distribution information is available for the core and filter/transition/shoulder materials (either from construction, specifications and/or borrow area investigations), then use the approach described in 8.1.4(a).
- If particle size distribution information is not available for the core and filter/transition/shoulder materials, then use the approach described in 8.1.4(b).

(a) Particle Size Distribution Information is Available

The recommended procedure involves the following steps:

Step 1: Adjust and select base soil gradings. Plot the particle size distributions for the core material and the filters or transitions which are protecting the core. If the maximum particle size of the core material is >4.75 mm, then re-grade the core grading such that the maximum size is 4.75 mm. If the base soil is gap graded, then re-grade the base soil grading on the particle size that is missing (i.e. at the point of inflection of the grading curve). Select representative gradings of the re-graded

Table A8.1 Conditional Probability Ranges for Continuation (Scenario 2).

Predominant Mode of Concentrated Leak	*Characteristics of downstream shoulder zone*	*Range of Conditional Probabilities for Continuing Erosion*
Cracking due to differential settlement (cross valley, foundation, embankment staging). Mechanism causing cracking in the core is also likely to cause cracking of the downstream shell (e.g. common cause cracking).	Well compacted, cohesive materials. Material likely to hold a crack.	1.0
	Poorly compacted, low plasticity materials. Material may collapse on wetting.	0.5–0.9
Desiccation cracking near crest, or on construction layer	Similar plasticity to core	0.5–1.0
	Lower plasticity than core, less prone to desiccation cracking	0.1–0.5
High permeability zone in the core or along the foundation contact, or Cracking due to differential settlement, features causing cracking in the core are not present below the downstream shell.	High permeability feature also likely to be present across the shoulder zone (e.g. shutdown surface)	0.5–1.0
	Leak unlikely to find an exit through the shoulder (i.e. very wide downstream shoulder, well compacted, low gradients, low erodibility, different compaction methods and lift thicknesses used in core and downstream shoulder)	0.01–0.1
	Leak likely to find an exit through the shoulder (e.g. narrow downstream shoulder, high gradient across shoulder, high erodibility, similar compaction methods and lift thicknesses used in core and downstream shoulder, materials placed in upstream/downstream orientation, feature extends part way through the shoulder)	0.1–0.5
Along outside of conduits passing through the dam	Leak also likely to be common cause through downstream shoulder (e.g. desiccation cracking on sides of excavation, poor compaction, arching in trench backfill)	0.5–1.0

base soil which are indicative of the finer 5% of the base soil gradings (fine base soil grading), the average grading (average base soil grading) and the coarser 5% of the base soils (coarse base soil grading). Figure A8.1 shows an example.

Step 2: Check for the blow out condition. In cases where there is limited depth of cover over the filter/transition zone, assess the potential for blow out by comparing the

seepage head at the downstream face of the core to the weight of soil cover. This is calculated as the ratio of the total stress from the vertical depth of soil (and rockfill) over the crack exit to the potential reservoir head. If the factor of safety is greater than about 0.5 three dimensional effects will be sufficient to make this a non-issue. If the factor of safety is less than about 0.1 it should be assumed the filter/transition will not be effective and probability of continuation $P_{CE} = 1.0$. Between these limits a probability of continuation between 0.1 and 0.9 should be applied. If the factor of safety for blow out is greater than 0.5, then follow the proceeding steps,

Step 3: Check if the filter/transition zone will hold an open crack. If the filter/transition zone contains an excess of silty or clayey fines, assess the potential for them to hold an open crack using Table A8.2. If the probability of the filter/transition holding a crack from Table A8.2 is ≥ 0.1, then evaluate the remaining steps by considering the 'cracked' filter/transition zone as the base soil and the zone downstream of the cracked filter as the filter material. This assumes the cracked filter zone will also erode. If the filter is cemented, then the 'cracked' filter zone should be ignored and the core evaluated against the zone downstream of the cracked filter.

Step 4: Check if the filter/transition zone is segregated. Assess the potential for segregation of the filter/transition/shoulder materials using Table A8.3, Table A8.4 and Table A8.5. If a continuous segregated layer is likely to be present, then estimate the grading of the segregated layer assuming that 50% of the finer soil fraction is segregated out leaving the remaining 50% of coarser fraction. Figure A8.2 shows a graphical method for adjusting the gradation curve to allow for segregation. Use the DF15 values from the adjusted grading curves for estimating the conditional probabilities of No Erosion, Some Erosion, Excessive Erosion and Continuing Erosion in the remaining steps.

Step 5: Check if the filter/transition zone is internally unstable. Evaluate the probability that the filter or transition zone materials are internally unstable (P_{IUS}) using Figure A3.4 for materials with >10% fines or Figure A3.5 for materials with <10% fines. If the probability of internal instability (P_{IUS}) is ≥ 0.3, then adjust the grading curve assuming that 50% of the unstable soil fraction is washed out. Figure A8.2 shows a graphical method for adjusting the gradation curve to allow for suffusion. Use the DF15 values from the adjusted filter grading curves for assessing the probability of No Erosion, Some Erosion, Excessive Erosion and Continuing Erosion in Step 7. If the probability of internal instability is <0.3, then do not adjust the filter grading curves.

Step 6: Evaluate the DF15 values for the No, Excessive and Continuing Erosion boundaries using Table A8.6 and Table A8.7 for the fine base soil grading, the average base soil grading and the coarse base soil grading. Plot the DF15 values for these boundaries on the grading curve limits of the filter/transition material (see Figure A8.4 for an example). Use the adjusted grading curves for the filter/transition zone if required to do so by the preceding steps 4 or 5.

Step 7: Estimate the probabilities for No Erosion, Some Erosion, Excessive and Continuing Erosion for each representative base soil grading. Estimate the proportion of the filter/transition gradings that fall into each of the particular erosion categories based on the plot of filter/transition grading curves versus Filter Erosion Boundaries (an example is shown in Figure A8.4). If there are no filter/transition

gradings that fall into the Continuing Erosion category, then use Table A8.8 to aid judgment in assigning probabilities for Continuing Erosion. This allows for the possibility of the gradations being coarser than indicated by the available information and depends on how much finer the gradings are to the Continuing Erosion boundary. The suggested approach is to estimate the proportions for the Some, Excessive and Continuing Erosion categories first (P_{SE}, P_{EE} and P_{CE}) and then calculate $P_{NE} = 1 - (P_{SE} + P_{EE} + P_{CE})$. Do this for each of the representative base soil gradings (fine, average and coarse gradings) as follows;

- For the fine base soil grading; P_{NE} fine, P_{SE} fine, P_{EE} fine, and P_{CE} fine.
- For the average base soil grading; P_{NE} ave, P_{SE} ave, P_{EE} ave, and P_{CE} ave.
- For the coarse base soil grading; P_{NE} coarse, P_{SE} coarse, P_{EE} coarse, and P_{CE} coarse.

Make an initial estimate of the probabilities of the No Erosion, Some Erosion, Excessive Erosion and Continuing Erosion branches by the sum-product of the % of base soil gradings and the % of NE, SE, EE and CE for each representative base soil grading. The calculations are as follows;

- $P_{NE} = (5\% \times P_{NE}\ \text{fine}) + (90\% \times P_{NE}\ \text{ave}) + (5\% \times P_{NE}\ \text{coarse})$.
- $P_{SE} = (5\% \times P_{SE}\ \text{fine}) + (90\% \times P_{SE}\ \text{ave}) + (5\% \times P_{SE}\ \text{coarse})$.
- $P_{EE} = (5\% \times P_{EE}\ \text{fine}) + (90\% \times P_{EE}\ \text{ave}) + (5\% \times P_{EE}\ \text{coarse})$.
- $P_{CE} = (5\% \times P_{CE}\ \text{fine}) + (90\% \times P_{CE}\ \text{ave}) + (5\% \times P_{CE}\ \text{coarse})$.

An example of summing of the probabilities is shown in Table A8.9 for the example shown in Figure A8.4.

Use judgment to adjust the calculated percentages to take into account the effects of other factors such as the distribution of the core and filter gradations in the fill, borrow area variability and selective placement of materials.

(b) Simplified Approach – Particle Size Distribution Information is Not Available

- Estimate the particle size distribution of the core materials based on the likely source of materials. The gradation of the soils may be able to be estimated based on the likely geological origin of the materials (e.g. decomposed granitic soils, residual soils, alluvial fine clays and silts, etc).
- Estimate the particle size distribution of the filter/transition/shoulder materials based on the likely source of the materials and whether they were processed or not (e.g. run-of-pit alluvial sands and gravels, unprocessed quarry fines or tunnel spoil, processed sand and gravels, etc). Estimate the DF15 of the filter/transition/downstream shoulder materials.
- Evaluate the DF15 values for the No, Excessive and Continuing Erosion boundaries for the estimated gradation of the core materials using Table A8.6 and Table A8.7.
- Estimate the probabilities for No Erosion, Some Erosion, Excessive and Continuing Erosion based on the estimated proportion of the filter/transition gradings that is likely to fall into each of the particular filter erosion categories. The suggested approach is to estimate the probabilities for Some, Excessive and Continuing Erosion (P_{SE}, P_{EE} and P_{CE}) and calculate $P_{NE} = 1 - (P_{SE} + P_{EE} + P_{CE})$.

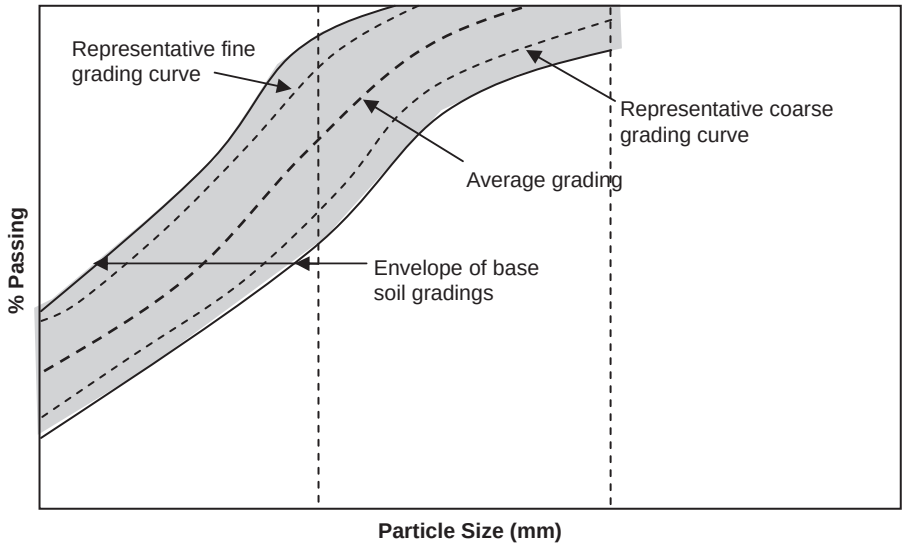


Figure A8.1 Example of the selection of representative grading curves (fine, average and coarse) for the assessment of filter compatibility.

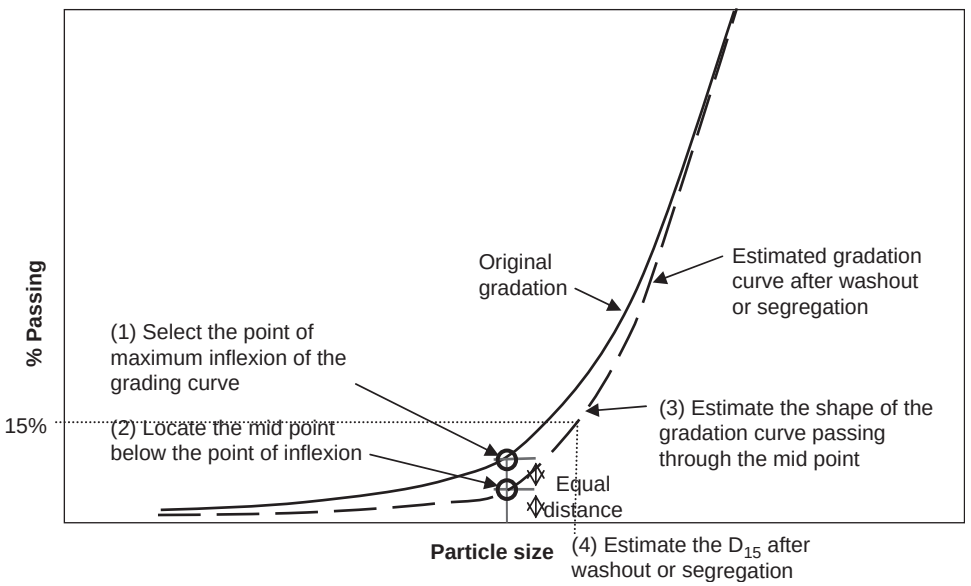

% Passing
15%
Original gradation
Estimated gradation curve after washout or segregation
(1) Select the point of maximum inflexion of the grading curve
(2) Locate the mid point below the point of inflexion
(3) Estimate the shape of the gradation curve passing through the mid point
Equal distance
Particle size
(4) Estimate the D_{15} after washout or segregation

Figure A8.2 Approximate method for estimating DF_{15} after washout of the erodible fraction from a suffusive soil or for soils susceptible to segregation.

Table A8.2 Likelihood for Filters with Excessive Fines Holding a Crack.

Fines Plasticity	*Fines Content % Passing 0.075 mm*	*Probability of holding a crack*	
		Compacted	*Not compacted*
Non plastic (and no cementing present)	5%	0.001	0.0002
	7%	0.005	0.001
	12%	0.05	0.01
	15%	0.1	0.02
	>30%	0.5	0.1
Plastic (or fines susceptible to cementing)	5%	0.05	0.02
	7%	0.1	0.05
	12%	0.5	0.3
	≥15%	0.9	0.7

Note: Fines susceptible to cementing for filters having a matrix predominately of sand sized particles (e.g. filters derived from crushed limestone).

Table A8.3 Potential for Segregation of Filtering Materials.

Factor	*Relative Importance Factor (RF)*	*Likelihood Factor (LF)*			
		Less Likely (1)	*Neutral (2)*	*More Likely (3)*	*Much More Likely (4)*
Construction practices	(3)	Good construction and stockpiling practices used	Fair construction and stockpiling practices	Poor construction or stockpiling practices	Very poor construction and stockpiling practices with no regard for segregation effects
		Placed in thin lifts < 0.6 m, careful control during construction.	End dumping from trucks, spread by dozer in thin lifts < 0.6 m	End dumping from trucks, spread by dozer in thick lifts > 0.6 m	Filters/transitions constructed by pushing material over the edge of the core
Gradation – Comparison to USBR/US Corps filter criteria, and % sand	(2)	Meets segregation criteria in Table 10.4.	Borderline segregation criteria in Table 10.4.	Fails segregation criteria in Table 10.4	Significant departure from segregation criteria in Table 10.4.
		>50% passing 4.75 mm sieve	>40% passing 4.75 mm sieve	25–40% passing 4.75 mm sieve	<25% passing 4.75 mm sieve
Width of zone	(1)	Wide zone, >6 m	3–6 m wide zone ()	Narrow zone, 1.5–3 m wide	Narrow zone, <1.5 m

Table A8.4 Gradation Limits to Prevent Segregation (USDA SCS 1994, USBR 1987, US Corps of Engineers 1994).

Minimum D10 (mm)	*Maximum D90 (mm)*
<0.5	20
0.5–1.0	25
1.0–2.0	30
2.0–5.0	40
5.0–10	50
10–50	60

Table A8.5 Susceptibility of filter/transition zones to segregation versus weighted score (Relative importance factor (RF)) x (Likelihood factor(LF)).

Weighted Score from Table A8.3	*Segregation Assessment*	*Consideration of Segregation Effects for Filter/Transition Assessment*
6–10	Low potential for segregation	Segregation of filter/transition materials do not need to be considered
11–17	Moderate potential for segregation	Segregation of filter/transition materials should be considered, unless investigations show otherwise.
18–24	High potential for continuous segregated layers	Segregation should be assumed to be present, unless investigations show otherwise.

(c) Estimate Potential Leakage Flows

Assess the potential leakage flows that could develop if piping were to initiate if piping were to initiate in the core based on the following guidance;

- Information from case histories of poor filter performance suggest the potential maximum leakage flows that could develop due to piping are as follows:
  - Filters falling into the Some Erosion category – up to 100 l/sec:
  - Filters falling into the Excessive Erosion category – 100 to 1000 l/sec.
  - Filters falling into the Continuing Erosion category – flows of >1000 l/sec) and increasing.

It is recommended that the maximum leakage flows listed above be used in the assessment of the probability of a breach mechanism developing. Lower leakage flows are likely if upstream flow limitation occurs, but the factors would need to be carefully considered and justified if they were to be relied on in the assessment.

Table A8.6 No erosion boundary for the assessment of filters of existing dams (after Foster and Fell 2001).

Base Soil Category	*Fines content (1)*	*Design Criteria of Sherard and Dunnigan (1989)*	*Range of DF15 for No Erosion Boundary From Tests*	*Criteria for No Erosion Boundary*
1	$\geq$ 85%	DF15 $\leq$ 9 DB85	6.4–13.5 DB85	DF15 $\leq$ 9 DB85 (2)
2	40–85%	DF15 $\leq$ 0.7 mm	0.7–1.7 mm	DF15 $\leq$ 0.7 mm (2)
3	15–40%	DF15 $\leq$ (40-pp% 0.075 mm) $\times$ (4DB85-0.7)/25 + 0.7	1.6–2.5 DF15 of Sherard and Dunnigan design criteria	DF15 $\leq$ (40-pp% 0.075 mm) $\times$ (4DB85-0.7)/25 + 0.7
4	<15%	DF15 $\leq$ 4 DB85	6.8–10 DB85	DF15 $\leq$ 4 DB85

Notes: (1) The fines content is the % finer than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.
(2) For highly dispersive soils (Pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower DF15 for the no erosion boundary. For soil group 1 soils, suggest use the lower limit of the experimental boundary, i.e. DF15 $\leq$ 6.4 DB85. For soil group 2 soils, suggest use DF15 $\leq$ 0.5 mm. The equation for soil group 4 would be modified accordingly.

Table A8.7 Excessive and Continuing erosion criteria (Foster 1999; Foster and Fell 1999, 2001).

Base Soil	*Proposed Criteria for Excessive Erosion Boundary*	*Proposed Criteria for Continuing Erosion Boundary*
Soils with DB95 < 0.3 mm	DF15 > 9 DB95	For all soils: DF15 > 9DB95
Soils with 0.3 < DB95 < 2 mm	DF15 > 9 DB90	
Soils with DB95 > 2 mm and fines content >35%	DF15 > the DF15 value which gives an erosion loss of 0.25 g/cm^2 in the CEF test (0.25 g/cm^2 contour line in Figure A8.3)	
Soils with DB95 > 2 mm and fines content <15%	DF15 > 9 DB85	
Soils with DB95 > 2 mm and fines content 15–35%	DF15 > 2.5 DF15 design, where DF15 design is given by: DF15 design = (35-pp%0.075 mm) (4DB85-0.7)/20 + 0.7	

Notes: Criteria are directly applicable to soils with DB95 up to 4.75 mm. For soils with coarser particles determine DB85 and DB95 using grading curves adjusted to give a maximum size of 4.75 mm.

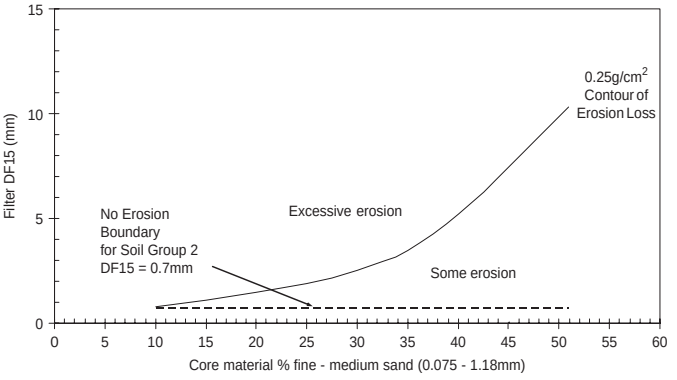
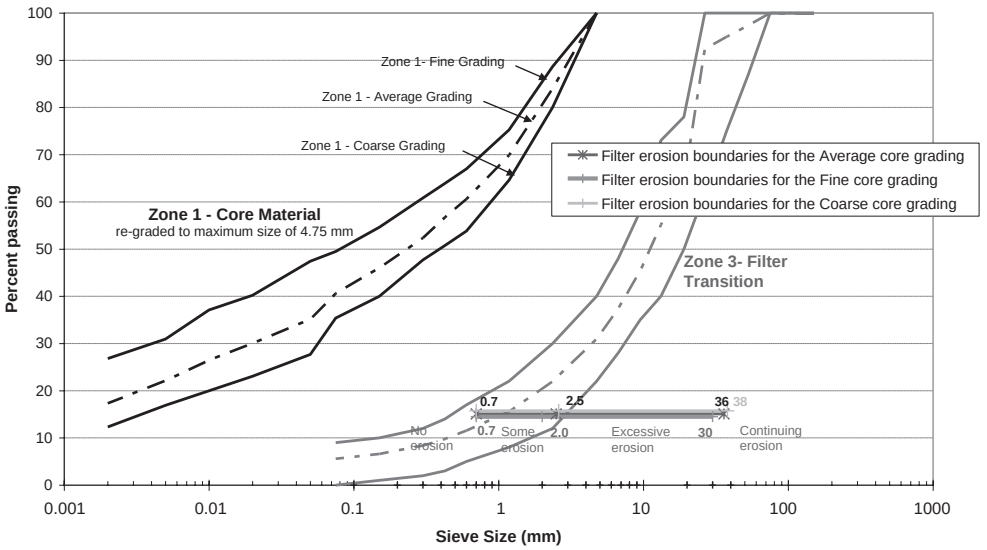


Figure A8.3 Criteria for excessive erosion boundary.

Table A8.8 Aid to judgment for estimation of probability for Continuing Erosion (P_{CE}) when the actual filter grading is finer than the Continuing Erosion Boundary.

Comparison of Actual DF15 of the Filter/transition Zone to the Continuing Erosion boundary	*Probability for Continuing Erosion (P_{CE})*
DF15 in dam $< 0.1 \times DF15_{CE}$	0.0001
DF15 in dam $< 0.2 \times DF15_{CE}$	0.001
DF15 in dam $< 0.5 \times DF15_{CE}$	0.01–0.05

Notes: $DF15_{CE}$ = DF15 for Continuing Erosion Boundary from Table A8.7.



Assessment of Zone 1 core against no erosion, excessive erosion and continuing erosion criteria

Core Gradation	Base soil sizes (mm)				No erosion	Excessive erosion	Continuing erosion
	DB85 (mm)	DB95 (mm)	% passing 0.075mm	% fine-medium sand (0.075 - 1.18mm)	DF15 (mm)	DF15 (mm)	DF15 (mm)
Fine Grading	1.9	3.3	50	25	0.7	2	30
Average	2.4	4	41	29	0.7	2.5	36
Coarse Grading	2.5	4.2	35	30	0.7	2.6	38

Figure A8.4 Example of plot showing filter/transition gradings compared to Filter Erosion Boundaries. Evaluate the filter erosion boundaries for the representative fine, average and coarse gradings of the core material.

A8.1.5 Probability for continuation – scenario 4 (internal erosion into an open defect, joint or crack in the foundation, in a wall or conduit)

For erosion to continue through an open defect, the defect needs to be sufficiently open to allow the soil surrounding the defect to pass through it. The recommended procedure is as follows:

- Evaluate the opening size that would allow Continuing Erosion (JOS_{CE}) of the surrounding soil using Table A8.10.

Table A8.9 Example of Estimating Probabilities for No, Some, Excessive and Continuing Erosion for the example shown in Figure A8.4.

	Estimated Proportion of Filter Gradings falling into each Filter Erosion category (from Figure 10.4)				
Representative Base Soil Grading	*No Erosion (NE)*	*Some Erosion (SE)*	*Excessive Erosion (EE)*	*Continuing Erosion (CE)*	*Sum P_{XX}*
Fine Base Soil Grading (represents 5% of finest grading curves)	P_{NE} fine = 20%	P_{SE} fine = 60%	P_{EE} fine = 20%	P_{CE} fine = 0%	100%
Average Base Soil Grading (represents 90% of grading curves)	P_{NE} ave = 20%	P_{SE} ave = 70%	P_{EE} ave = 10%	P_{CE} ave = 0%	100%
Coarse Base Soil Grading (represents 5% of coarsest grading curves)	P_{NE} coarse = 20%	P_{SE} coarse = 70%	P_{EE} coarse = 10%	P_{CE} coarse = 0%	100%
Calculation of Probabilities for No, Some, Excessive and Continuing Erosion (P_{xx})	$P_{NE} = (5\% \times P_{NE}$ fine$) + (90\% \times P_{NE}$ ave$) + (5\% \times P_{NE}$ coarse$)$	$P_{SE} = (5\% \times P_{SE}$ fine$) + (90\% \times P_{SE}$ ave$) + (5\% \times P_{SE}$ coarse$)$	$P_{EE} = (5\% \times P_{EE}$ fine$) + (90\% \times P_{EE}$ ave$) + (5\% \times P_{EE}$ coarse$)$	$P_{CE} = (5\% \times P_{CE}$ fine$) + (90\% \times P_{CE}$ ave$) + (5\% \times P_{CE}$ coarse$)$	
Calculation Result	20%	69.5%	10.5%	0%	100%
Assigned Probabilities	$P_{NE} = 0.20$	$P_{SE} = 0.69$	$P_{EE} = 0.11$	$P_{CE} = 0.0001$ [(a)]	1.000

Notes: (a) Even though there are no filter gradings falling into the Continuing Erosion category in this example, a probability of 0.0001 was assigned for Continuing Erosion based on the guidance given in Table A8.8. This takes into account the possibility of the materials in the dam being coarser than indicated by the gradation curves. In this example, the filter gradation envelope is significantly finer than the Continuing Erosion boundary, and hence a very low probability is assigned based on Table A8.8.

Table A8.10 Continuing Erosion criteria for erosion into an open defect.

	Comparison of Soil Gradation to Joint/Defect opening size (JOS)
Erosion condition	*All Soils*
Continuing erosion (CE)	$JOS_{CE} = D_{95}$ surrounding soil

Notes: • JOS_{CE} = Joint/defect opening size that would allow continuing erosion of the surrounding soil.
• D95 should be based on the average soil grading after re-grading on 4.75 mm particle size.

- Estimate the conditional probability for Continuing Erosion by estimating the proportion of soil gradations that are coarser than the Continuing Erosion category and using Table A8.11.

Table A8.11 Aid to judgment for estimation of probability for continuation for open defects/joints/cracks.

Comparison of joint opening in the dam (JOS) to the Continuing Erosion criteria	
Joint Opening in the dam	*Probability for Continuing Erosion (P_{CE})*
$JOS < 0.1 \times JOS_{CE}$	0.0001
$JOS < 0.2 \times JOS_{CE}$	0.001
$JOS < 0.5 \times JOS_{CE}$	0.01
$JOS > JOS_{CE}$	0.1–0.9 Estimate based on the proportion of gradings finer than CE

Notes: JOS_{CE} = Joint Opening for Continuing Erosion Boundary = D95 of the surrounding soil.

A8.1.6 Probability for continuation – scenario 5 (erosion into a Toe Drain)

This scenario is applicable if the failure path under consideration involves a seepage path that exits into a toe drain which could lead to continuing erosion of the embankment or foundation materials.

Table A8.12 Probability of continuation for erosion into toe drains.

Drain Inspection	*Design and Construction Details*	*Probabilities for Continuing Erosion into Toe Drains*
Video inspections indicate no deterioration or damage to the pipes	Good design and construction details (e.g. filter surround present, proper pipe)	0.0001 to 0.0005
	Poor design and construction details (e.g. thin plastic pipe, geotextile surround)	0.0005 to 0.005
No video inspection of drain, and no external evidence of poor performance (i.e. clear outflow, no sinkholes)	Good design and construction details (e.g. filter surround present, good joint details) and deterioration unlikely (metal pipe <20 years)	0.0005 to 0.005
	Poor design and construction details (e.g. thin plastic pipe, geotextile wrap of drainage gravel). Or deterioration likely to be present (e.g. metal pipe >30 years, old tile drains, nearby source of tree roots)	0.005 to 0.05
Video inspection of drain shows. broken pipe, open joints, materials found in drain) Or external evidence of sinkholes over drain, evidence of internal erosion in outflow)		0.1–0.9 depending on severity of damage 0.1 to 0.5 if some damage but no actual surrounding material has moved into pipes 0.5 to 0.95 if large openings in pipe and surrounding material has moved into pipe

The assessment of erosion into a toe drain considers the observed condition of the toe drain (from video or external inspections) and the design and construction details of the toe drain. Estimate the probability of continuing erosion for erosion into a toe drain using Table A8.12.

A8.2 PROBABILITY FOR CONTINUATION FOR INTERNAL EROSION THROUGH THE FOUNDATION

A8.2.1 Approach

The probability of continuation of erosion should be estimated by:

a) Assessing the probability that the exit will be a filtered or unfiltered exit.
b) Given the exit is unfiltered the probability of continuation will be 1.0.
c) Given the exit is filtered, estimate the probability for Continuing Erosion using the method described in Section A8.1.4.

The probability of continuation will be the product of the probability of an unfiltered exit and the probability assessed considering the filters.

A8.2.2 Probability of filtered or unfiltered exit

The likelihood that the exit will be filtered or unfiltered will depend on the failure path being considered, the embankment zoning and the details of the foundation geology. The steps are as follows;

- Estimate the probability that there will an unfiltered exit (P_{unf}) using Table A8.13 to aid judgment. The conditional probability of continuation for this scenario is equal to (P_{unf}) × 1.0.
- Calculate the probability that there will be a filtered exit (P_{fe}) = 1 − P_{unf}. Assess the filter materials and materials being eroded in terms of the Continuing Erosion criteria using the procedure in Section A8.1.4.
- The probability of continuing erosion $P_{CE} = (P_{unf} \times 1.0) + (P_{fe} \times P_{CE})$.

Note that in situations where a foundation filter drain or toe drain system is present, there is still the possibility that seepage paths could bypass the filter drain system due to the geological conditions, compaction of soil foundations below foundation filters, or deteriorated toe drain systems. Examples of these scenarios are included in Table 8.13 and illustrated in Figures A8.5 to A8.7).

It needs to be recognized that low permeability strata beneath horizontal drains may prevent them working effectively, Figure A8.6 shows an alluvial foundation where the lower permeability strata (A and E) will prevent the seepage in the most permeable sand and gravel strata (B and D) from flowing into a filtered exit in the horizontal drain.

For the situation of backward erosion piping within a foundation sand layer which has an overlying low permeability layer, the likelihood of finding an unfiltered exit is already considered in the assessment of initiation of backward erosion. An unfiltered exit is implicit if a heave condition or a sand boil is present.

Table A8.13 Probability of by-passing the foundation filter for piping through the foundation or piping from the embankment into the foundation.

Scenarios	*Examples*	*Range of Probabilities of the Seepage Path Bypassing the Foundation Filter*
Foundation filter fully penetrates the foundation material that is erodible Filter drain has adequate discharge capacity	Filter trench completely penetrates through the foundation sand/silt layer, in filled defects in rock or desiccated foundation clay and into a non-erodible material (Figure A8.5a)	Negligible Assume probability unfiltered exit = 0
Foundation filter partially penetrates the foundation material that is erodible Filter drain has adequate discharge capacity	Filter trench penetrating through foundation sand/silt layer, in filled defects in rock or desiccated foundation clay but erodible soil or in filled defects in rock remains beneath the filter trench (Figure A8.5b)	0.01 to 0.001
Foundation filter partially penetrates the foundation material that is erodible Filter drain has adequate discharge capacity	Filter trench partially penetrating through foundation sand/silt layer, or desiccated foundation clay but some of this soil or in filled defects in rock remains beneath the filter trench (Figure A8.5b)	0.1 to 0.01
Foundation filter partially penetrates the foundation material that is erodible Filter drain has inadequate discharge capacity	Filter trench partially penetrating through foundation sand/silt layer, or desiccated foundation clay (Figure A8.5b)	0.1 to 0.9 depending on the degree to which filter drain capacity is exceeded
Foundation filter blanket drain directly overlying the foundation material that is erodible, clean-up of the foundation allows seepage into the filter drain	Filter drain overlying a foundation sand/silt layer, desiccated foundation clay (Figure A8.5c) or in filled defects in rock.	0.01 – 0.001 depending on the width and discharge capacity of the filter drain. For very narrow widths the probability could be as high as 0.1
Foundation filter blanket drain directly overlying the foundation material that is erodible, filter drain has inadequate discharge capacity and or clean-up of the foundation hinders flow of seepage into the filter drain	Filter drain overlying a foundation sand/silt layer, desiccated foundation clay (Figure A8.5c) or in filled defects in rock.	0.5 – 0.01 depending on the width of the filter drain, clean-up of the foundation to allow seepage to flow into the filter drain and discharge capacity of filter drain
Low permeability layer is present below the foundation filter drain. Filter drain has adequate discharge capacity	Low permeability soil layer between the foundation filter/toe drain and the foundation material that is being eroded (e.g. Figure A8.6) Residual soil left over jointed bedrock below the foundation filter	0.1–0.5

(*Continued*)

Table A8.13 Continued.

Scenarios	*Examples*	*Range of Probabilities of the Seepage Path Bypassing the Foundation Filter*
Scarifying and rolling the foundation below the foundation filter in soils with macrostructure, Filter drain has adequate discharge capacity	Foundation soils or highly weathered rocks containing relict defects, root holes, desiccation cracks, or lateritic soils.	0.1–0.5
High permeability soil or rock has an unprotected exit downstream	Soil layer that can be piped daylights downstream of the dam Continuous open jointed rock daylights downstream of the dam (Figure A8.7)	0.5–0.9
Discharge capacity of foundation filter drainage system is not sufficient	Thin sand filter layer on high permeability soil or rock foundation Filter materials contain excess of fines (>5% fines passing 0.075 mm)	0.1–0.9
Blocked or collapsed toe drains	HDPE corrugated pipes, collapsed tile drains, biological growth in pipes	0.1–0.9
No foundation filter or toe drain system provided, cohesive shell materials		1.0

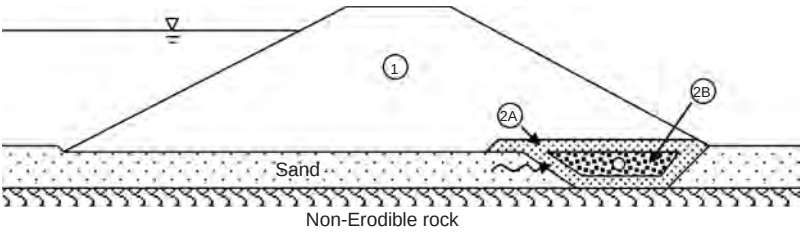
A8.3 PROBABILITY OF CONTINUATION FOR INTERNAL EROSION OF THE EMBANKMENT AT OR INTO THE FOUNDATION

A8.3.1 Erosion into open joints in rock foundation

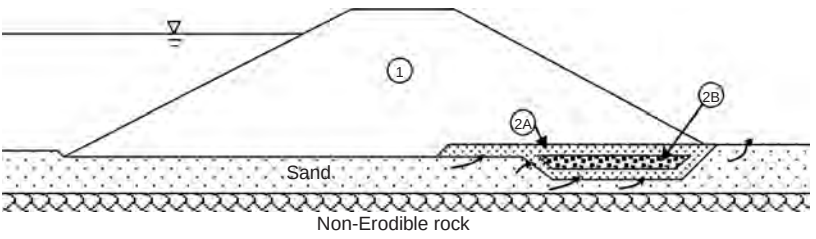
Evaluate the probability of the joint openings being sufficiently open and continuous to allow erosion of the core materials based on the outcomes of the assessment in Sections A5, A6 and A7. Use the method described in Section A8.1.5 to evaluate the probability of continuing erosion.

If the geological conditions of the rock mass are such that the open joints at the core-foundation contact are unlikely to daylight at an unfiltered exit, or need to exit via an interconnected path of another joint set which are less open, then reduce the probability for continuing erosion using the judgment probability mapping tables in Appendix E in the Piping Toolbox.

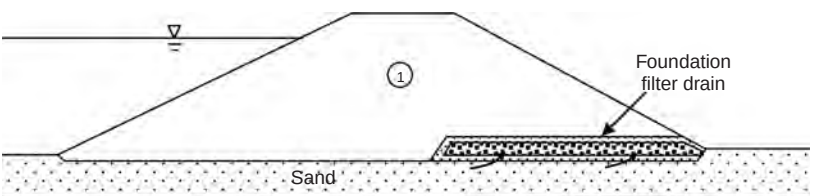
If the geological conditions are such that the seepage through the rock mass needs to exit through an overlying soil layer (e.g. alluvium overlying a jointed rock mass), then evaluate the probability of continuing erosion at both the contact with the rock defects and also at the contact with the overlying soils. The assessment of continuation



(a) Example of fully penetrating filter drain



(b) Example of partially penetrating filter drain



(c) Example of horizontal filter drain directly overlying erodible foundation material

Figure A8.5 Examples of scenarios of fully penetrating and partially penetrating foundation filter drains.

at the soil layer contact needs to consider the potential volume of embankment material that may be stored within the open jointed rock mass before filtering takes place.

A8.3.2 Erosion into Coarse Grained Soil Foundation

Evaluate the probability of the coarse grained soil foundation layer being sufficiently open and continuous to allow erosion of the core materials. Evaluate the probability of the foundation soils permitting Continuing Erosion of the core materials using the methods described in Section A8.1.4.

If the geological conditions of the soil layering are such that the open soil layers at the core-foundation contact are unlikely to daylight at an unfiltered exit, or need to exit via an interconnected path of another soil layer which is capable of acting as an effective filtering medium, then reduce the probability for continuing erosion using the judgment probability mapping tables in Appendix E in the Piping Toolbox.

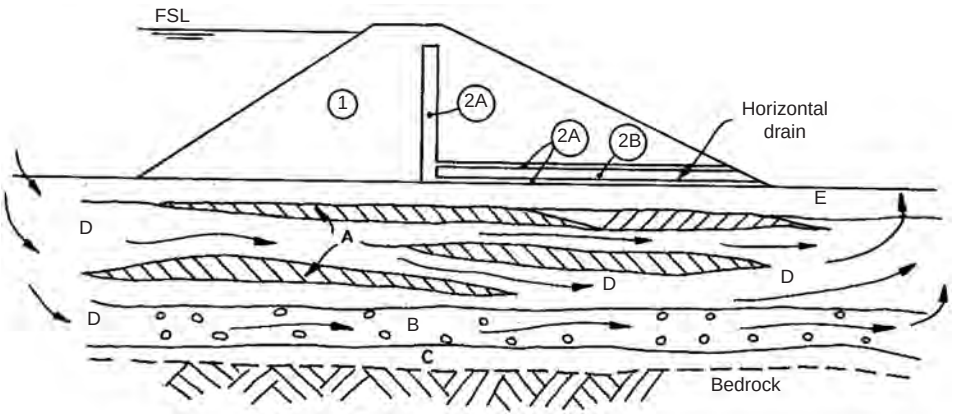


Figure A8.6 Example of an embankment where much of the seepage flow will be to an unfiltered exit.

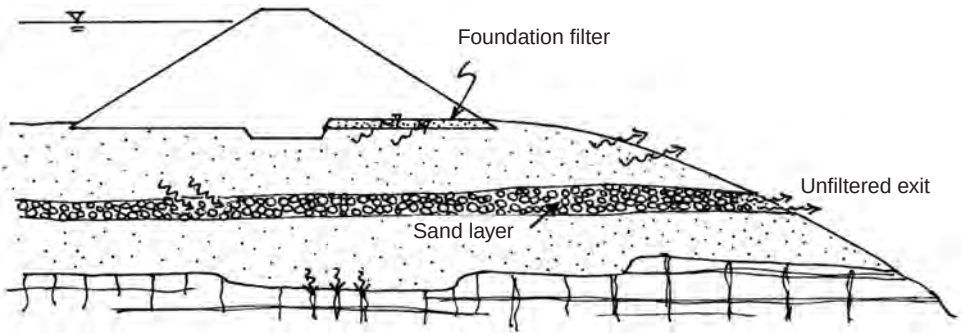


Figure A8.7 Example of an embankment where there is an unfiltered exit due to day lighting of the foundation sand layer downstream of the dam.

A9 Probability of Progression

See Section A11 of the Piping Toolbox

A10 Probability of Detection, Intervention and Repair

See Section A12 of the Piping Toolbox.

A11 Probability of Breach

See Section A13 of the Piping Toolbox.

Geotechnical Engineering of Dams, 2nd edition provides a comprehensive text on the geotechnical and geological aspects of the investigations for and the design and construction of new dams and the review and assessment of existing dams. The main emphasis of this work is on embankment dams, but much of the text, particularly those parts related to geology, can be used for concrete gravity and arch dams.

All phases of investigation, design and construction are covered. Detailed descriptions are given from the initial site assessment and site investigation program through to the preliminary and detailed design phases and, ultimately, the construction phase. The assessment of existing dams, including the analysis of risks posed by those dams, is also discussed. This wholly revised and significantly expanded 2nd edition includes a lengthy new appendix on the assessment of the likelihood of failure of dams by internal erosion and piping.

This valuable source on dam engineering incorporates the 200+ years of collective experience of the authors in the subject area. Design methods are presented in combination with their theoretical basis, to enable the reader to develop a proper understanding of the possibilities and limitations of a method. For its practical, well-founded approach, this work can serve as a useful guide for professional dam engineers and engineering geologists and as a textbook for university students.

Photograph front:
Hinze Dam at completion of Stage 3 construction.
Courtesy of The Hinze Dam Alliance (comprising Seqwater, SKM, Thiess and URS).

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